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TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOLUME 91

December, 1927

Published by the American Society of Civil Engineers
33 West Thirty-ninth Street, New York

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VOLUME 91
DECEMBER, 1927

Edited by the Secretary, under the direction of the Committee on Meetings and Publications.

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PUBLISHED BY THE SOCIETY

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Page 280, line 24 from top, for "Mr. Scheidenhelm questions the existence of uplift within the dam", read "Mr. Scheidenhelm questions the relative importance of uplift within the dam".

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Paper No. 1622

STRAIGHT LINE PLOTTING OF SKEW FREQUENCY DATA*

By R. D. Goodrich,† M. Am. Soc. C. E.

With Discussion by Messrs. H. Alden Foster, A. Streiff, W. A. Shewhart, W. H. R. Nimmo, John Tucker, Jr., L. Standish Hall, E. P. Goodrich, Arne Fisher, Allen Hazen, and R. D. Goodrich.

Synopsis

Valuable and suggestive papers have been presented to the Society in recent years on methods for the investigation of the frequency of rainfall and run-off of various intensities, for use in engineering design.

This paper deals with the application of new equations for both the "Integral" or "Duration" Curve and the "Frequency" or "Differential" Curve; and for expressing the variations in the occurrence of phenomena which follow "Skew Frequency Laws" as to their distribution. These equations are particularly applicable to the plotting of hydraulic data (rainfall and run-off) having a limited number of observations in the series.

The equation of the Duration Curve is more easily obtained and is of simpler form than that of the Frequency Curve; it is important to note, however, that with either equation given the other can be written at once. Fractional exponents are to be expected, but they do not affect the fixed relations between the two equations. The equations of the Duration Curve are such that a curve can always be plotted as a straight line function of data that follow various skew frequency laws. One general equation is given, which can be simplified for the condition of finite or infinite limits.

Numerous examples of the use of these equations in the plotting of curves on a new type of skew frequency paper are shown. Ordinary and logarithmic

* Presented at the meeting of September 8, 1926.

† Engr. in Chg. of Works, Chihli River Comm., Tientsin, China.

co-ordinate paper may also be used. These curves cover a greater variety of conditions than have heretofore been presented. Curves are plotted with equal facility when either or both the coefficient of variation and coefficient of skew are negative as well as when the mode is greater than the mean. The same plotting paper is used when either one or both the limits are finite. The equations can also be applied to the representation of certain population curves.

The equations and methods can be used by any engineer, as the work is largely graphical and most of the difficulties and objections to the use of curves and equations based on statistical methods of analysis are obviated.

Introduction

For several years the Chihli River Commission has been engaged in studying flood conditions in the Province of Chihli, North China, but great difficulty has been experienced in securing data as to rainfall and run-off, especially regarding flood flows.

In order to make the fullest use of the meager data available, all the recent papers and discussions which have appeared in the publications of the Society have been studied with great care and with valuable results. While more records covering longer periods of observation are greatly to be desired, the methods of utilizing the existing short-term records are being constantly improved by engineers throughout the world. It is with the hope of making another step forward in this direction and of adding something to the value and usefulness of the shorter series of observations which are becoming more generally available that the following method is presented for treating skew frequency data so that they can be plotted according to a "straight line" law of variation.

The difficulties and objections to the use of methods developed for frequency studies of statistical data suggested a new angle for attacking the problem of applying frequency curves to engineering. The following entirely empirical equations and the methods developed for their use are the result of several years' study consisting of a search for equations that could be used as guides in estimating the magnitude and frequency of floods, and in developing and extending the equations when found. A skew frequency paper having logarithmic co-ordinates was devised. On this paper original data may be plotted and then replotted as a straight line function for several different skew frequency laws of distribution, irrespective of the signs of the "coefficient of variation" and "coefficient of skew", and also for any value of the "skewness" whether it be positive or negative. The methods are here presented in the hope that they will prove as useful to other engineers as they have to the writer.

Mathematics of Skew Frequency Equations

Nomenclature.—

Let t = serial number of records when arranged in order of magnitude, or the percentage of the time.

n = number of records in the series, or 100 per cent.
R = value of any record in the series.
a = lower finite limit of the curve representing the records (not the lowest record).
b = upper finite limit of the curve representing the records (not the highest record).
c, d = exponents of $f(R)$, values of which are found from the curve.
h, k = coefficient of $f(R)$, values of which are found from the curve; but $h = kM$.

$$M = \log_{10} e = 0.43429\,; \quad \frac{1}{M} = 2.30259.$$

$\Gamma\left(\frac{1}{c} + 1\right)$ = gamma function of $\left(\frac{1}{c} + 1\right)$.

r = radius of gyration of the area under the frequency curve about the t' axis.
sv = standard variation.
cv = coefficient of variation.
cs = coefficient of skew.

$$y = \text{colog}\left(1 - \frac{t}{n}\right).$$

The equations are distinguished by letters indicating the application of the particular form. For example, the general formula is labeled Equation (G), and the special form, Equation (F), etc. The designation, Equation (A), signifies that this form has the a-limit; Equation (B) has both the a-limit and the b-limit. Equation (A') is the differential of Equation (A) and Equation (A''), etc. The subscripts indicate variations in the base equations, as, for example, Equation (A_1) is a special form of Equation (A).

General Equation.—The general equation* of the integral or duration curve is:

$$t = n - n\,(10)^{-h\frac{(R-a)^c}{(b-R)^d}}$$

or,

$$t = n - n\,e^{-k\frac{(R-a)^c}{(b-R)^d}} \qquad (G)$$

In this equation it is evident that when $R = a$, $t = 0$; also when $R = b$, $t = n$ and, hence, n is the length of the ordinate at the end of the curve where the tangent (or asymptote) is parallel to the R-axis.

Special Forms of the Equation.—In some cases the equation may be conveniently used in the following forms:

$$t = n\,(10)^{-h\frac{(R-a)^c}{(b-R)^d}}$$

or,

$$t = n\,e^{-k\frac{(R-a)^c}{(b-R)^d}} \qquad (F)$$

* This equation was derived by an entirely empirical process and subsequently compared with those of Pearson, Kapteyn, and others which were not known to the writer at the time

When both c and d are positive, the curve for this equation is the reverse of that for Equation (G). That is, the curve begins above the t-axis at the height, n, and the R-axis becomes the tangent or asymptote at the right-hand end of this curve. If c and d are negative, the curve begins on the R-axis as in Equation (G).

In practice, Equation (F) may be used with c and d positive, if t is defined as the percentage of time for which any given value of R may be exceeded. Numerical computations are reduced by using this equation when plotting curves of rainfall or run-off on ordinary or logarithmic co-ordinate paper. However, the ordinates to the differential or frequency curve derived from this equation would be negative; this might seem incorrect although in reality it matters but little. If the frequency curve is derived from Equation (G), the ordinates will be positive, and, hence, for convenience in the mathematical investigations, the second form (Equation (G)) will be discussed. When $d = 0$, Equation (G) becomes:

$$t = n - n\,(10)^{-h\,(R-a)^c}$$

or,

$$t = n - n\,e^{-k\,(R-a)^c} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

In Equation (A) the lower limit of R is a, which may be either positive, negative, or zero, while the upper limit is ∞, and it may be used, therefore, in place of Types III, V, and VI of the Pearson curves.*

When $a = 0$, it becomes a special case and is in the simplest form, that is,

$$t = n - n\,(10)^{-h\,R^c}$$

or,

$$t = n - n\,e^{-k\,R^c} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (S)$$

If Equation (S) had been derived from Equation (F) it would be in the form of Equation No. XV, in "Empirical Equations", by Professor Running, a study of which stimulated the investigation that finally led to the development of the general form.

When d and c are unequal, Equation (G) is too complicated for general use;† but when $d = c$, while a and b are both finite, it takes the following form which may be used in place of Type I of the Pearson curves:

$$t = n - n\,(10)^{-h\left[\frac{R-a}{b-R}\right]^c}$$

or,

$$t = n - n\,e^{-k\left[\frac{R-a}{b-R}\right]^c} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

When $c = 0$, Equation (G) becomes:

$$t = n - n\,(10)^{\frac{-h}{(b-R)^d}}$$

or,

$$t = n - n\,e^{\frac{-k}{(b-R)^d}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (C)$$

The curve for Equation (C) has $-\infty$ as its lower limit and b as its upper limit. This type of curve does not appear to be included in any of Professor

* The several types of curves here referred to are shown in *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 179, 182.

† The form with $d = 2c$ is useful in studies of certain population statistics.

Pearson's types of skew frequency equations while the writer's equations do not include his Types II, IV, and VII. In Equation (F), when $d = 0$ and c is negative, the lower limit is a and the upper limit is $+\infty$. If $d = c$ (both negative) the limits are finite and it takes the form of Equation (B), but with $\frac{R-a}{b-R}$ inverted. It may be stated, therefore, that the equations thus far given cover the entire practical range of skew frequency curves as fully as the Pearson equations. The general equation, (G), contains too many constants for ordinary use and a limited experience shows that Equations (A), (B), and (S) are sufficient for frequency investigations of most rainfall and run-off data.

Although Equation (C) can be discussed in a manner similar to that given subsequently for Equations (S) and (A), it does not apply to the conditions met in ordinary hydraulic problems and, therefore, it will not be considered further.

Frequency Curves.—The general equation of the differential or frequency curve in this system of representation is obtained from Equation (G) by differentiation, thus,

$$t' = \frac{d\,t}{d\,R} = n\,k\,\frac{(R-a)^{c-1}}{(b-R)^{d+1}}\,[c\,(b-R) + d\,(R-a)]\,e^{-k\,\frac{(R-a)^e}{(b-R)^d}} \ldots (G')$$

From this, the equations of the frequency curves corresponding to the integral Equations (S), (A), and (B) are obtained by substituting the proper values of d and a, as before:

$$t' = n\,c\,k\,R^{(c-1)}\,e^{-k\,R^c} \ldots\ldots (S')$$

$$t' = n\,c\,k\,(R-a)^{c-1}\,e^{-k\,(R-a)^e} \ldots\ldots (A')$$

$$t' = n\,c\,k\,(b-a)\,\frac{(R-a)^{c-1}}{(b-R)^{c+1}}\,e^{-k\left[\frac{R-a}{b-R}\right]^e} \ldots\ldots (B')$$

Equation (S') is strikingly similar to Equation (8)* presented by H. Alden Foster, Assoc. M. Am. Soc. C. E., with R^e in place of z in the exponent of e. Since Equation (G') was derived by differentiation from Equation (G), and is entirely general, the total area under the curve of Equation (G') will always equal the maximum ordinate, n, of Equation (G) as for other systems of frequency curves. These equations, therefore, evidently represent forms of skew frequency curves; the particular form of curve depending on the character of $f(R)$ and the values of the constants, k and c.

To continue the examination further, differentiate Equation (G'). Then:

$$t'' = n\,k\,\frac{(R-a)^{c-1}}{(b-R)^{d+1}}\,[c\,(b-R) + d\,(R-a)]\,e^{-k\,\frac{(R-a)^e}{(b-R)^d}}$$

$$\left\{\frac{d-c}{c\,(b-R) - d\,(R-a)} + \left[\frac{c-1}{R-a} + \frac{d+1}{b-R}\right] - k\,\frac{(R-a)^{c-1}}{(b-R)^{d+1}}\right.$$

$$\left.\left[c\,(b-R) + d\,(R-a)\right]\right\} \ldots\ldots (G'')$$

This equation may similarly be simplified to the following three forms:

$$t'' = n\,c\,k\,R^{c-2}\,[(c-1) - c\,k\,R^c]\,e^{-k\,R^c} \ldots\ldots (S'')$$

* "Theoretical Frequency Curves and Their Application to Engineering Problems," *Trans-*

$$t'' = n\,c\,k\,(R-a)^{c-2}\,[(c-1) - c\,k\,(R-a)^{c}]\,e^{-k\,(R-a)^{c}} \ldots\ldots (A'')$$

$$t'' = n\,c\,k\,\frac{(R-a)^{c-2}}{(b-R)^{c+2}}\left[(c-1) + \frac{2\,(R-a)}{b-a} - c\,k\left(\frac{R-a}{b-R}\right)^{c}\right]$$

$$(b-a)^{2}\,e^{-k\left(\frac{R-a}{b-R}\right)^{c}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B'')$$

In these equations it is evident that when $f\,(R) = \infty$, $t'' = 0$ and hence the frequency curves become tangent to the R-axis at the right-hand end.

When $f(R) = 0$, there are several different conditions which depend upon the value of c:

1.—If $c > 2$, $t'' = 0$ and the frequency curve is tangent to the R-axis at the left. (See Figs. 1 and 2.)

2.—If $1 < c < 2$, $t'' = \infty$ and the frequency curve is perpendicular to the R-axis at the left; but for all three equations the curve begins on the R-axis. (See Figs. 3 and 7.)

3.—If $c = 2$, the frequency curves will leave the R-axis at a definite angle.

4.—If $c = 1$, all the frequency curves will start at a definite height above the R-axis, equal to $n\,k$ and at an angle the tangent of which is $-n\,k^2$. The duration curve, therefore, will leave the R-axis at a definite angle. Furthermore, in this case all values of t'' will be negative and there will be no hump in the frequency curve.

5.—If $c < 1$, $t'' = \infty$ and the frequency curve begins at the left with the ordinate through a as tangent or asymptote, and as all values of t'' will be negative, there will be no hump in the frequency curve. The duration curve will begin perpendicular to the R-axis. (See Figs. 6, 8, and 10.)

6.—In Equation (F), if $d = 0$ or c, and c is negative, the frequency curve begins and ends on the R-axis. In this case the first and second derivatives are identical with those for Equation (9). (See Fig. 11.)

L. Standish Hall, Assoc. M. Am. Soc. C. E., has called attention to these types of frequency curves.* In Mr. Hall's paper, however, only the first type is considered, while the writer's equations and methods of plotting apply to all these types as well as to those in which the maximum point is to the right of the center of gravity of the area under the curve. (See Figs. 4 and 5.) All these types are illustrated in the diagrams. The type of frequency curve is usually determined by the value of c which is the slope of the auxiliary line obtained by this method of plotting. (Fig. 9 is an exception to this rule since the mode is less than the mean while $c > 3.3$.)

Location of Mode and Mean.—When c is greater than unity, the value of R for the maximum point on the frequency curve, that is, the mode, is obtained by placing the expression within the brackets in the equations for t'' equal to zero. This results in the following:

$$\text{Mode}_S = \left(\frac{c-1}{c\,k}\right)^{\frac{1}{c}};\quad \text{Mode}_A = a + \left(\frac{c-1}{c\,k}\right)^{\frac{1}{c}} \ldots\ldots\ldots (1)$$

* See Fig. 12 of his paper entitled "The Probable Variations in Yearly Run-off as Determined from a Study of California Streams," *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 240.

The mode for Equation (B') is not so easily obtained. It is that value of R which satisfies the following expression:

$$\frac{R-a}{b-R} = \left[\frac{c-1}{c\,k}\,\frac{2\,(R-a)}{c\,k\,(b-a)}\right]^{\frac{1}{c}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

This expression can be solved only approximately and the following is the best the writer has been able to obtain. In Equation (2) substitute R' for $R - a$. The equation may then be written:

$$\frac{\frac{R'}{b-a}}{1-\frac{R'}{b-a}} = \left(\frac{c-1}{c\,k}\right)^{\frac{1}{c}} \left\{1 + \frac{2}{c\,(c-1)}\,\frac{R'}{(b-a)}\right\}^{\frac{1}{c}}$$

Expand the left-hand member by simple division, divide both sides of the equation by the first factor of the right-hand member and then expand the second factor by the binomial theorem. The result is:

$$\left(\frac{c\,k}{c-1}\right)^{\frac{1}{c}} \left[\frac{R'}{b-a} + \left(\frac{R'}{b-a}\right)^2 + \left(\frac{R'}{b-a}\right)^3 + \left(\frac{R'}{b-a}\right)^4 + \ldots\ldots\ldots\ldots\right]$$
$$= 1 + \frac{2}{c\,(c-1)}\left(\frac{R'}{b-a}\right) - \frac{2}{c^2\,(c-1)}\left(\frac{R'}{b-a}\right)^2 + \frac{4\,(2\,c-1)}{c^3\,(c-1)^2}\left(\frac{R'}{b-a}\right)^3$$
$$- \frac{2\,(2\,c-1)\,(3\,c-1)}{3\,c^4\,(c-1)^3}\left(\frac{R'}{b-a}\right)^4 + \ldots\ldots\ldots\ldots\ldots (3)$$

By collecting the coefficients of corresponding powers of R' and discarding terms with the third and higher powers, a quadratic is obtained which can be solved for R'. The value of R as obtained from this will be approximately equal to the mode. The result will probably be 2 or 3% too small. By computing the coefficients for one or more of the higher powers of R' and solving by methods for incommensurate roots, the results could be made as accurate as desired.

The mean for a frequency curve has been defined as the abscissa of the center of gravity of the area under the curve. Hence:

$$\text{Mean} = \frac{\int R\,t'\,d\,R}{\int t'\,d\,R}$$

Although the writer has not been able to solve this integral for the case when both limits of the frequency curve are finite, it can be solved for Equations (S') and (A'), in which R varies from zero or a to ∞.

The method will be shown for Equation (S'). Substitute the value of t' from Equation (S') in the integral; then:

$$\text{Mean}_S = \frac{\int_0^\infty n\,c\,k\,R^c\,e^{-k\,R^c}\,d\,R}{\int_0^\infty n\,c\,k\,R^{c-1}\,e^{-k\,R^c}\,d\,R} = c\,k\int_0^\infty R^c\,e^{-k\,R^c}\,d\,R$$

This follows since the integral of the denominator is the total area under the frequency curve, which is n. Now, let $R^c = x$, then,

$$d\,R = \frac{1}{c}\,x^{\frac{1}{c}-1}\,d\,x$$

Hence:

$$\text{Mean}_S = k \int_0^\infty x^{\frac{1}{c}} e^{-k x} \, d x$$

This is a standard form of definite integral which can be evaluated in terms of the gamma function:*

$$\text{Mean}_S = k^{\frac{-1}{c}} \Gamma\left(\frac{1}{c} + 1\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Similarly, the mean for Equation (A') is:

$$\text{Mean}_A = k^{\frac{-1}{c}} \Gamma\left(\frac{1}{c} + 1\right) + a \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Standard Deviation and Skewness.—If the square of the "standard variation" of a series is defined as equal to the square of the radius of gyration of the area under the frequency curve for the given series, about the center of gravity of that area, it can be determined for Equation (S') by integrating in the same manner as was done for the mean. In integral form, the square of the radius of gyration of the area under the curve about the t'-axis is:

$$r^2 = \frac{\int R^2 t' \, d R}{\int t' \, d R} = c \, k \int_0^\infty R^{c+1} \, e^{-k R^c} \, d R$$

This becomes:

$$r^2 = k^{\frac{-2}{c}} \Gamma\left(\frac{2}{c} + 1\right)$$

The square of the standard variation is found by subtracting the square of the mean, that is, the distance from the t'-axis to the center of gravity of the area under the curve. Hence:

$$(c \, v)^2 = k^{\frac{-2}{c}} \left[\Gamma\left(\frac{2}{c} + 1\right) - \Gamma\left(\frac{1}{c} + 1\right)^2\right] \ldots\ldots\ldots\ldots\ldots (6)$$

This formula applies also to Equation (A') as the abscissa of the point of beginning of the curve is eliminated in the process.

The coefficient of variation is the standard variation divided by the mean, and, therefore, for Equations (S') and (A'), it is:

$$(c \, v)_S = \frac{\left[\Gamma\left(\frac{2}{c} + 1\right) - \Gamma\left(\frac{1}{c} + 1\right)^2\right]^{\frac{1}{2}}}{\Gamma\left(\frac{1}{c} + 1\right)} \ldots\ldots\ldots\ldots\ldots (7)$$

$$(c \, v)_A = \frac{\left[\Gamma\left(\frac{2}{c} + 1\right) - \Gamma\left(\frac{1}{c} + 1\right)^2\right]^{\frac{1}{2}}}{a \, k^{\frac{1}{c}} + \Gamma\left(\frac{1}{c} + 1\right)} \ldots\ldots\ldots\ldots\ldots (8)$$

With Equations (6), (7), and (8), it is now possible to compute the "skewness" for the same equations, by the relation:

$$\text{Skewness} = \frac{\text{Mean} - \text{mode}}{\text{Standard variation}} \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

* An excellent table of $\log \Gamma (p)$ is given by Mr. Foster, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), Table 1, pp. 152–153.

While it is possible thus to solve the integral for r^2 for the "third moment" about the t'-axis, the writer has not been able to do so for the "gravity axis" and hence the "coefficient of skew", as used by Allen Hazen, M. Am. Soc. C. E., Mr. Foster and others, has not been determined.*

Equations (6), (7), and (8) show that the "skewness" of curves fitted by Equation (A), depends only on the value of the exponent, c. It varies inversely as this exponent and is a maximum when the mode is equal to a, that is, when $c = 1$. If c is less than unity the mode is imaginary, but practically it remains the same as for $c = 1$. The skewness becomes zero when,

$$\Gamma\left(\frac{1}{c}+1\right) = \left(1-\frac{1}{c}\right)^{\frac{1}{c}}$$

from which, $c = 3.3121$.

As previously stated, the value of c is determined from the slope of the straight line drawn through the data when plotted as described for the auxiliary curve. Hence, for all data fitted by Equations (S) and (A) the slope of the line indicates at once whether the skewness of the frequency curve is positive or negative.

It does not seem to be possible to develop suitable formulas for the equation for curves with both limits finite, corresponding to the foregoing coefficients and functions used in connection with systems of skew frequency curves developed by statistical methods. However, the mode, mean, standard variation, coefficient of variation, and doubtless other functions of these curves, can all be found by graphical methods for any given case. For example, after having completed the straight line plotting of the data, the curve may be replotted with ordinary rectangular co-ordinates. From this curve, by plotting the first and second differential curves, the intersection of the second curve with the R-axis will of course give the mode.

The duration curve is the first integral of the frequency curve and, therefore, its ordinates give areas under the frequency curve. By plotting a second integral curve, moments are obtained from which the position of the center of gravity of the area under the frequency curve can be determined, giving the mean. In the same way, the radius of gyration can be obtained after plotting a third integral curve. Sufficient data, therefore, can be obtained by graphical means for calculating the same functions of the curve with finite limits, as were obtained mathematically for the other curves.

In addition to the difficulty of expressing the functions of a new system of skew frequency curves in terms of an entirely different system derived by radically different methods, and designed for different uses, it seems to the writer that the coefficients taken from these older systems have little, if any, physical significance when applied to ordinary hydraulic problems.

One or two functions of these curves will be suggested, which can be easily calculated with data obtained from the duration curve. These functions will

* While the term, "coefficient of skew," will occasionally be used in the following text, it should be understood to denote the "skewness" as measured in Equation (9), except where the context clearly indicates otherwise. It should be noted that Equation (9) gives a positive or negative value according to whether the mean is greater or less than the mode: it is, therefore, more indicative of the true form of the curve than the "coefficient of skew" derived from the sum of the squares of the variations from the mean.

give a ready means of comparing curves for different streams or localities and will also call attention to conditions which should be given consideration in designing engineering works.

Factors for Classifying Frequency Curves.—When the foregoing equations are used for the investigations of skew frequency data especially as applied to hydraulic problems, the following functions or quantities may be used for the purpose of comparing different curves as well as for obtaining general information about an individual curve. The first ones proposed may be called the "average range" and the "average frequency" of the curves or data.

The average range is obtained by taking from the duration curve as plotted by any of the foregoing methods, the values of R (expressed as a percentage of the mean when desired for comparison with other curves) for 25% and for 75% of the time. The difference of these values may be called the "range of average run-off" (rainfall, floods, etc.), or briefly, the "average range" for the given stream or locality. This range is such that 50% of the records will lie within its limits. That the record for the next succeeding year will come within this range is just as probable as that it will fall outside. It is evident that the area under the frequency curve between the ordinates corresponding to these two points on the duration curve, is one-half the total area under the curve, and hence if the data followed the "normal law of error" this quantity would be equal to twice the "probable error of the series". In general, the average range varies directly as the coefficient of variation for the series. One-half the average range is given as one of the measures of dispersion by G. Udny Yule,* who designates it the "quartile deviation" or "semi-interquartile range". Attention is called to the fact that this "is not a measure of the deviation from any particular average". Yule states that "for distributions approaching the ideal forms [of normal or moderately skew curves] the semi-interquartile range is usually about two-thirds of the standard deviation", and, therefore, that nine times the semi-interquartile range, or four and one-half times the average range, will approximately cover 99% of the total frequency. He also states that this measure of dispersion has the clearest and simplest meaning of any of the three which he describes, and that it is calculated with the greatest ease.

The average frequency of the average range of the data, or briefly the average frequency, may be obtained by dividing one-half the total area under the frequency curve by the percentage covered by the average range. This gives the average height of the frequency curve between the limits of this range. If the range is small, the average frequency will be high, that is, greater than unity, and *vice versa*. This is illustrated in Table 1, where these factors are given for several streams, together with the coefficients of variation as published for the same stream.

In addition to the formula for the skewness of a frequency curve as used by Pearson (the mean minus the mode divided by the standard deviation), Yule* gives another measure of skewness, and one which can be easily com-

* "An Introduction to the Theory of Statistics," by G. Udny Yule, Fourth Edition, Chapter VIII, p. 147.

puted with quantities taken from the duration curve. This might be called the "quartile ratio" to distinguish it. It is obtained from values of the variable corresponding to 25%, 50%, and 75% of the time. If these quantities are, respectively, $Q_{0.25}$, $Q_{0.50}$, $Q_{0.75}$, then:

$$\text{Quartile ratio} = \frac{(Q_{0.75} - Q_{0.50}) - (Q_{0.50} - Q_{0.25})}{\frac{1}{2}(Q_{0.75} - Q_{0.25})} = \frac{Q_{0.25} + Q_{0.75} - 2\,Q_{0.50}}{\frac{1}{2}(Q_{0.75} - Q_{0.25})}$$

This ratio becomes zero for a symmetrical or nearly symmetrical distribution, "and while its highest possible value is 2, it would rarely in practice attain higher numerical values than 1." When this quantity is positive, the mode will be less than the mean, while for negative values the reverse will be true. In Table 1, values of this ratio are given for the streams in Column (5), and these may be compared with the coefficients of skew given in Column (6).

TABLE 1.—Comparison of Coefficients and Factors.

Name of stream. (1)	Average range, percentage. (2)	Average frequency. (3)	Coefficient of variation. (4)	Quartile* ratio. (5)	Skewness. (6)
Richelieu River, New York	21	2.38	0.16	0.00	−0.02
West Canada Creek, New York	35	1.43	0.21	+0.14	0.60
Sacramento River, California	38	....	0.29	−0.11	0.14
Rock Creek, California	44	1.15	0.21	−0.09	....
Merrimac River, Massachusetts	49	1.02	0.24	+0.13	−0.01
Kings River, California	59	0.85	0.41	+0.22	0.41
Mackenzie River, Australia	62	0.81	0.44	−0.07	−0.22
Yuba River, Colorado	65	0.77	0.40	−0.22	....
Kern River, California	68	0.74	0.56	+0.33	1.23
Donner Creek, California	76	0.66	0.50	−0.05	....
Sweetwater River, California	97	0.52	1.87	+0.63	3.66

* Quartile ratio $= \frac{\text{Abnormal range}}{\text{Subnormal range}}$, discussed later.

Preparation of Skew Frequency Paper

In order to use the foregoing equations for straight line plotting of data in the form of integral or duration curves, they must be put into logarithmic form. Equation (S) will be used as an illustration, but the transformations are equally applicable to the more general forms. By transposing to the left-hand member of the equation the terms containing n, and then dividing both sides by $-n$, it becomes;

$$1 - \frac{t}{n} = (10)^{-h\,R^c}$$

This can now be transformed into the logarithmic form:

$$\log_{10}\left(1 - \frac{t}{n}\right) = -h\,R^c, \text{ or } \operatorname{colog}_{10}\left(1 - \frac{t}{n}\right) = h\,R^c$$

Since n is always greater than t, $\frac{t}{n}$ will always be less than unity and $1 - \frac{t}{n}$ will also be less than unity. Hence, $-\log\left(1 - \frac{t}{n}\right)$, or $\operatorname{colog}\left(1 - \frac{t}{n}\right)$, will

be a positive quantity. Let this be represented by y. The equation now becomes;

$$y = h\,R^c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (S_1)$$

This equation is in the exponential form and, therefore, represents a straight line when plotted on logarithmic paper. Similarly, Equations (A) and (B) can be written in the logarithmic form, indicated by the subscript, unity, as follows:

$$y = h\,(R - a)^c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A_1)$$

$$y = h\left(\frac{R - a}{b - R}\right)^c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B_1)$$

In Equation (A_1), when y is plotted to R, the curve will be concave upward if the constant, a, is positive (see Figs. 2, 3, and 4) and will be concave downward if this constant is negative (see Fig. 5). However, when the correct value of a is determined, y and $R - a$ will plot on a straight line. When $f(R)$ takes the form given in Equation (B_1), y plotted to R will take various forms depending on the signs used. In most cases a will be positive giving the minus sign in the numerator, which with the minus sign before R in the denominator, will give reverse curves like those of Figs. 7 and 8. If a is negative, giving a plus sign in the numerator, while the sign of R in the denominator is also plus, the reverse curve will be like that of Fig. 9. If the sign in the numerator becomes minus while that in the denominator is plus, the curve will be sharply concave upward, the right-hand end seeming to approach some vertical line as an asymptote (see Fig. 10). If these two signs are reversed the left-hand end will approach a vertical asymptote in the downward direction (see Fig. 17) and the whole curve will be reversed. When correct values of both a and b have been determined, y and the function, $\dfrac{R - a}{b - R}$, will plot as a straight line.

Equation (S_1) can evidently be transformed into another logarithmic form;

$$\log_{10} y = \log_{10} h + c \log_{10} R \ldots\ldots\ldots\ldots\ldots\ldots\ldots (S_2)$$

This is in the form of the slope equation of a straight line and hence if log y is plotted to log R on ordinary cross-section paper, the points will lie on a straight line the slope of which, with respect to the log R-axis, is c and the intercept of which on the log y-axis is log h. It is possible, therefore, to determine the values of the constants, c and h, by this method of plotting. The values of these constants can also be obtained by solving Equation (S_2) after forming simultaneous equations, by substituting for log y and log R the co-ordinates of two points on the line. This method is frequently to be preferred to the purely graphical method, especially when logarithmic or skew frequency paper is being used.

Equations (A_1) and (B_1) also can be written in similar form:

$$\log_{10} y = \log_{10} h + c \log_{10} (R - a) \ldots\ldots\ldots\ldots\ldots\ldots (A_2)$$

$$\log_{10} y = \log_{10} h + c \log_{10} \left(\frac{R - a}{b - R}\right) \ldots\ldots\ldots\ldots\ldots\ldots (B_2)$$

When log y is plotted to log R, the curves for Equations (A_2) and (B_2) will have the same forms as when Equations (A_1) and (B_1) are plotted on logarithmic paper, or when Equations (A) and (B) are plotted on skew frequency paper.

To make a skew frequency paper on which corresponding values of the percentage of the time, t, and rainfall or run-off, R, or f (R), will plot as linear functions, it is only necessary to take the ordinary logarithmic ruling in one direction for the R and f (R)-scales, and then compute a suitable series of values of log colog $\left(1 - \frac{t}{n}\right)$ to give the required spacing for the t-scale. An example of such ruling is given in Fig. 12. All the other diagrams have been plotted on this kind of paper. The origin for the spacing of the t-scale is the 90% line. This becomes evident since $\left(1 - \frac{90}{100}\right) = 0.1$. The colog of 0.1 is unity and the log of unity is zero. Hence, for percentages greater than 90, the ordinates laid off to the right as the log colog will be positive, while for percentages less than 90, the values of log colog will be negative and the ordinates must be laid off to the left of that line.

Although the computation of suitable spacings is not at all difficult if a table of cologarithms is available, Table 2 may be convenient in laying out this skew frequency paper and its use will avoid uncertainties and errors. Table 2 is easily modified or extended as desired.

TABLE 2.—Spacing of Ordinates for Skew Frequency Paper.

t = percentage of time.

x = distance from origin for skew frequency ruling.

t.	$+x$.	t.	$-x$.	t.	$-x$.	t.	$-x$.	t.	$-x$.
99.99	0.602	90	0.000	50	0.521	10	1.339	0.6	2.583
99.98	0.568	88	0.036	48	0.547	9	1.387	0.5	2.662
99.95	0.519	86	0.068	46	0.573	8	1.441	0.45	2.708
99.9	0.477	84	0.099	44	0.599	7	1.501	0.4	2.759
99.8	0.431	82	0.128	42	0.626	6	1.571	0.35	2.818
99.6	0.380	80	0.156	40	0.654	5	1.652	0.3	2.886
99.4	0.347	78	0.182	38	0.683	4.5	1.699	0.25	2.963
99.2	0.322	76	0.208	36	0.713	4	1.751	0.2	3.060
99	0.301	74	0.233	34	0.744	3.5	1.811	0.15	3.186
98	0.230	72	0.251	32	0.776	3	1.878	0.1	3.363
97	0.183	70	0.282	30	0.810	2.5	1.959	0.05	3.664
96	0.146	68	0.305	28	0.846	2.0	2.057	0.04	3.759
95	0.114	66	0.329	26	0.883	1.8	2.103	0.03	3.886
94	0.087	64	0.353	24	0.924	1.6	2.155	0.02	4.061
93	0.063	62	0.376	22	0.967	1.4	2.213	0.01	4.363
92	0.040	60	0.400	20	1.014	1.2	2.281	.....	.....
91	0.020	58	0.424	18	1.064	1.0	2.361	.....	.....
90	0.000	56	0.448	16	1.121	0.9	2.406	.....	.....
.....	.....	54	0.472	14	1.184	0.8	2.457	.....	.....
.....	.....	52	0.496	12	1.256	0.7	2.516	.....	.....

Application of Formulas to Specific Data

Hydraulic Data of the Usual Types.—It has been possible to plot satisfactory frequency curves for nearly all the hydraulic data which have come

to the writer's notice by the use of Equations (A), (B), and (S). Equation (C) and forms involving other functions of R have not been investigated thus far.

Several diagrams and tables are given to illustrate the great variety of frequency curves included, and to enable one who is not familiar with such curves to become so by making use of the data given. The data have been selected to illustrate as many types of skew frequency distributions as possible, in most part from hydraulic records. Some data from other sources have been included to illustrate particular forms. A possible application of the equations to population curves will be indicated subsequently as this does not seem to be strictly a problem in frequencies.

The work of plotting diagrams is greatly simplified by the use of the skew frequency paper as it will accommodate curves having any coefficient of variation and any degree of skewness, either or both of which may be positive or negative.

Fig. 1 shows the curves for the frequency distribution of the annual run-off of the Sacramento River, at Red Bluff, Calif., as given in Table 3, Column (1). These and other data were taken from Mr. Hall's paper previously mentioned. They were first arranged in the order of increasing magnitude. The percentages in Column (2) opposite each item in Column (1), were computed by taking the series of odd numbers 1, 3, 5 . . . . . $(2n - 3)$, $(2n - 1)$, and dividing each by $2n$, n being the number of records in the series.* Corresponding quantities from Columns (1) and (2) were then plotted as shown in Fig. 1, and a straight line drawn so as to average the points as nearly as possible. It is seen that the points follow this line, and the line therefore may be taken as the integral or duration curve for the skew frequency distribution of the total annual discharge of this river.

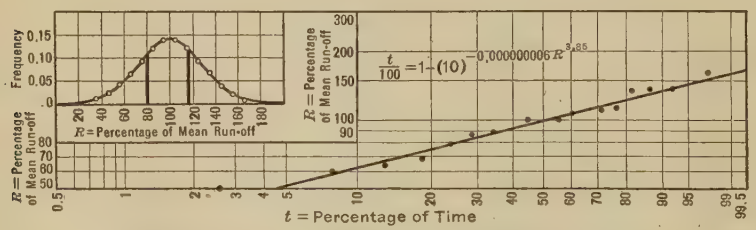

FIG. 1.—CURVES FOR SACRAMENTO RIVER AT RED BLUFF, CALIF.

Where such a short series of records lie so close to a straight line, it is not necessary to plot any other curves or make any but the simplest computations to arrive at conclusions which may be adopted as guides in engineering reports and designs. However, to make clearer the characteristics of this curve, the data of the last three columns are given, from which the frequency or differential curve was drawn. The percentages of time in Column (4) were read

* This method using the series of odd numbers is equivalent to finding the percentage at the center of each block of a histogram, whereas the simple finding of the ratio in percentage which each item bears to the whole would give the corresponding percentage of the larger valued side of each block. Furthermore, by the latter method, the last value would correspond with 100% which cannot be used in any usual exponential formula.

from the curve for the quantities in Column (3), these being taken with the common difference, $\Delta R = 10$ per cent. The quantity, $\frac{\Delta t}{\Delta R}$, was then set down in Column (5), from which the frequency curve, shown in the corner of the Fig. 1, was drawn. The lower limit of this curve is zero while the upper limit is infinity. This is indicated by the fact that a straight line may be used to represent the data as originally plotted. Hence, Equation (*S*) can be used to express the law of frequency variation. From the duration curve it is found that for 25% of the time the run-off will probably be less than 80% of the mean, while for 75% of the time it will be less than about 118 per cent.

TABLE 3.—Frequency Data for Sacramento River, at Red Bluff, California.*

Run-off as percentage of the mean = *R*.	Percentage of time, *t*.	*R*, read from the curve.	*t*, read from the curve.	$\frac{\Delta t}{\Delta R}$	Run-off as percentage of the mean = *R*.	Percentage of time, *t*.	*R*, read from the curve.	*t*, read from the curve.	$\frac{\Delta t}{\Delta R}$
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
50.3	2.6	....	....	....	103.3	55.3	120	77.0	0.93
58.8	7.9	30	0.6	0.13	103.3	60.5	130	86.3	0.67
64.6	13.1	40	1.9	0.26	107.4	65.8	140	93.0	0.37
69.3	18.4	50	4.5	0.45	112.5	71.1	150	96.7	0.21
78.3	23.7	60	9.0	0.65	113.4	76.3	160	98.8	0.08
86.4	28.9	70	15.5	0.95	135.9	81.6	170	99.6	....
89.5	34.2	80	25.0	1.20	136.8	86.9	....	....	....
90.4	39.5	90	37.0	1.40	138.9	92.1	....	....	....
99.0	44.7	100	51.0	1.37	161.2	97.4	....	....	....
100.6	50.0	110	64.7	1.28					

* Data from Table 1, *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 194–196.

Therefore, the run-off will be from 80 to 118% of the mean for 50% of the time. In other words, there is an even chance that the next succeeding record will fall within these limits.

As the maximum point of the frequency curve is so near the 100% ordinate, the mode is nearly equal to the mean and the skewness is small. The abscissa of the median as determined from the duration curve is the intersection of the curve with the "50%-of-time" ordinate, and it is seen to be about 99 per cent. That is, approximately one-half the area under the frequency curve lies on each side of the 99% ordinate.

The range of the average run-off, that is, the range of values which will probably occur about one-half the time, is indicated by the two heavy ordinates to the frequency curve. Their position is determined by finding the intersection of the duration curve, with the 25% and 75% time ordinates. The nearly symmetrical character and the small skewness of the curve shows that, for any given range on either side of these limits, the probabilities of high

and low annual discharge are about equal. These conclusions can also be drawn from the slope of the duration curve or from its equation, which is:

$$t = 100 - 100(10)^{-0.00000006\,R^{3.85}}$$

The exponent, $c = 3.85$, is determined, as has been shown, by the slope of the line with reference to the R-axis. When the slope of the line (or the value of c) is between the values of 2.5 and 4.5 approximately, the skewness will be comparatively small, while high and low records will have nearly equal frequencies. This can also be found easily from the duration curve without determining its equation. Thus, 5% of the time a value of 52% of the mean run-off will not be exceeded, while 95% of the time the run-off will probably be not greater than 145% of the mean. The quantity, $80 - 52 = 28$, is the variation in the value of the mean between 5 and 25% of the time, while the variation in the mean from 75 to 95% of the time is $145 - 118 = 27$. When the ratio of the ranges of these extremes is nearly unity, as in this case, high and low records are about equally probable.

All these conclusions as to the probable range of average conditions and the relative frequency of high and low discharges or run-off, together with the approximate values of each, which are to be expected in perhaps 50 to 100 years, are easily determined from Fig. 1 without the computation of any coefficients, factors, or criteria. Annual maximum flood heights and discharges often seem to follow this simple "linear" frequency law, making the method particularly applicable to the study of flood frequencies and heights.

TABLE 4.—Annual Rainfall at Tientsin, China.*

Year.	Rainfall, in inches, R.	Percentage of time, t.	$R - 8$.	Year.	Rainfall, in inches, R.	Percentage of time, t.	$R - 8$.
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
1902	10.0	1.6	2.0	1915	19.5	51.6	11.5
1920	11.1	4.7	3.1	1918	19.9	54.7	11.9
1900	12.3	7.8	4.3	1919	20.0	57.8	12.0
1899	12.4	10.9	4.4	1892	21.4	60.9	13.4
1922	13.2	14.1	5.2	1916	21.6	64.1	13.6
1895	14.4	17.2	6.4	1893	22.4	67.2	14.4
1909	14.7	20.3	6.7	1901	23.2	70.3	15.2
1913	15.4	23.4	7.4	1896	23.6	73.4	15.6
1906	15.6	26.6	7.6	1904	23.6	76.6	15.6
1898	15.9	29.7	7.9	1897	24.4	79.7	16.4
1903	16.0	32.8	8.0	1911	25.7	82.8	17.7
1907	16.2	35.9	8.2	1908	26.3	85.9	18.3
1917	16.3	39.1	8.3	1914	26.6	89.1	18.6
1910	17.8	42.2	9.8	1891	28.3	92.2	20.3
1921	19.2	45.3	11.2	1894	30.5	95.3	22.5
1905	19.5	48.4	11.5	1912	31.3	98.4	23.3

* Data from records of the British Municipal Council, Tientsin, China.

Table 4 gives the data used in plotting the curves for annual precipitation at Tientsin, China, shown in Fig. 2; and Table 5 the annual run-off, in second-feet per square mile, for the Allegheny River at Red House, N. Y. (Fig. 3). Both these curves are expressed by Equation (A) in which the lower limit is greater than zero. The original data are first plotted as was done for Fig. 1, but in Figs. 2 and 3 the data shown by the solid dots are such

that a smooth curve drawn as an average of the points will be concave upward. In order to extend such a curve beyond the limits of the actual observations the data must be treated so that when replotted they will approximate a straight line. This auxiliary line may then be extended and from it a smooth curve can be plotted graphically through the points as first platted. The necessary treatment is easily and rapidly effected by subtracting a constant quantity from each record of rainfall or run-off as the case may be.

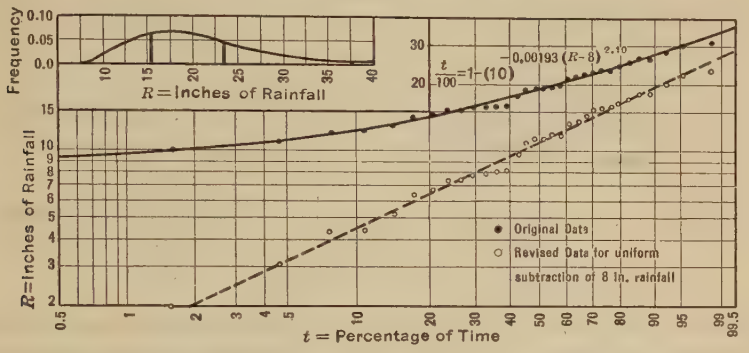

FIG. 2.—CURVES FOR ANNUAL RAINFALL AT TIENTSIN, CHINA.

The constants selected after two or three trials were 8 in. to be subtracted from the precipitation records (Fig. 2) and 0.075 sec-ft. per sq. mile for the run-off records (Fig. 3). The results are shown in Column (4) of Table 4 and Column (3) of Table 5. These quantities were then plotted against their corresponding percentages of time as indicated by the open circles in Figs. 2 and 3. If the constant selected for the first trial is too small, the points will lie along a curve which is flatter than the original one, that is, between the dotted straight lines and the curves. If too large a quantity is used, the auxiliary curve will be reversed, so as to be concave downward and to lie below the position of the dotted straight line.

TABLE 5.—RUN-OFF OF ALLEGHENY RIVER AT RED HOUSE, NEW YORK.*

Run-off, in second-feet per square mile, R.	Percentage of time, t	$R - 0.075$.	Run-off, in second-feet per square mile, R.	Percentage of time, t.	$R - 0.075$
(1)	(2)	(3)	(1)	(2)	(3)
0.085	3.0	0.01	0.135	55.9	0.06
0.085	8.8	0.01	0.155	61.8	0.08
0.105	14.7	0.03	0.16	67.7	0.085
0.11	20.6	0.035	0.165	73.5	0.09
0.115	26.5	0.04	0.185	79.4	0.11
0.13	32.3	0.045	0.195	85.3	0.12
0.12	38.2	6.045	0.24	91.2	0.165
0.125	44.1	0.05	0.245	97.0	0.17
0.13	50.0	0.055			

* Data from Fig. 10, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 170.

Smooth curves can now be plotted through the original points by means of the auxiliary dotted lines. For example (Fig. 2), this curve crosses the line

for a value of $R - 8 = 8$, on the ordinate for $t = 30$ per cent. On the same ordinate locate a point eight units above, that is, on the line for $R = 16$.

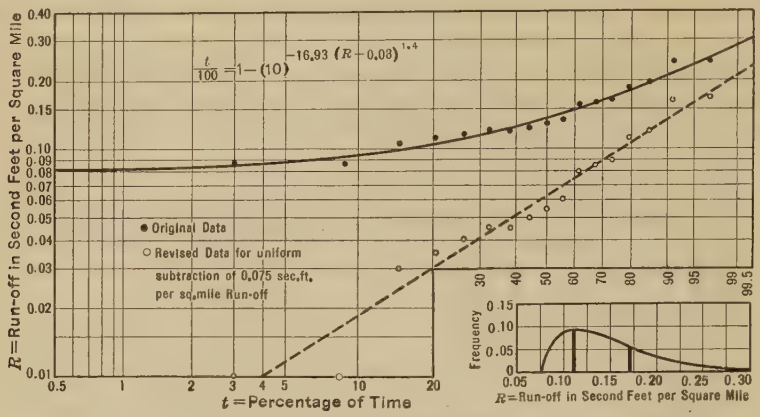
$\frac{t}{100} = 1 - (10)^{-16.93\,(R+0.08)^{1.4}}$
R=Run-off in Second Feet per Square Mile
t=Percentage of Time
● Original Data
○ Revised Data for uniform subtraction of 0.075 sec.ft. per sq.mile Run-off
Frequency
R=Run-off in Second Feet per Square Mile

FIG. 3.—CURVES FOR RUN-OFF OF ALLEGHENY RIVER AT RED HOUSE, N. Y.

This will be a point on the required curve. In the same manner any number of points may be plotted so as to extend the curve to any required distance in either direction. All the curves of Figs. 2 to 6, inclusive, were plotted in this manner.

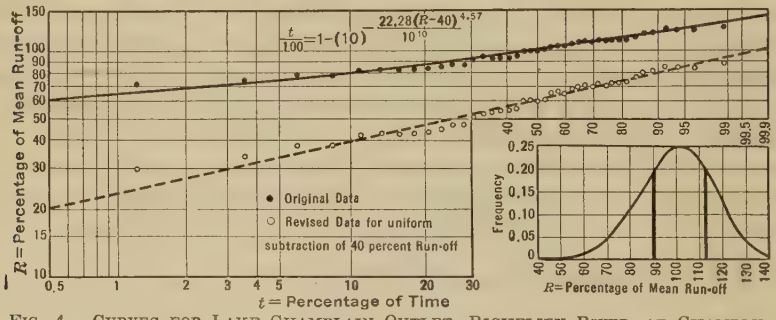
$\frac{t}{100} = 1 - (10)^{-\frac{22.28(R-40)^{4.57}}{10^{10}}}$
R=Percentage of Mean Run-off
t=Percentage of Time
● Original Data
○ Revised Data for uniform subtraction of 40 percent Run-off
Frequency
R=Percentage of Mean Run-off

FIG. 4.—CURVES FOR LAKE CHAMPLAIN OUTLET, RICHELIEU RIVER, AT CHAMBLY, QUE., CANADA.

The duration curves in Figs. 2 and 3 are similar, the only apparent difference being in the degree of curvature, Fig. 2 being the flatter. However, there is a marked difference in the left-hand part of the frequency curves. In Fig. 2 this curve begins tangent to the R-axis, while in Fig. 3 it is perpendicular to this axis. This difference is also evident from the equations of the curve. In the first case the value of the exponent, c, is greater than 2, while in the second case it is between 1 and 2. The change takes place at the value, 2, for which value the curve would leave the axis at some definite angle. These characteristics have been demonstrated in the mathematical analysis of the equations. It should be noted that the equations for the curves of Figs. 1, 4, and 5, have values greater than 3.3 for the exponent, c, and, therefore, that in each case the mode is greater than the mean, giving negative skewness.

TABLE 6.—Richelieu River Outlet of Lake Champlain, at Chambly, Quebec, Canada.*

Percentage of mean run off, R.	Percentage of time, t.	$R - 40$.	Percentage of mean run-off, R.	Percentage of time, t.	$R - 40$.	Percentage of mean run off, R.	Percentage of time, t.	$R - 40$.	Percentage of mean run-off, R.	Percentage of time, t.	$R - 40$.
70	1.2	30	87	25.6	47	100	50.0	60	111	74.4	71
74	3.6	34	87	28.1	47	101	52.5	61	112	76.8	72
78	6.1	38	91	30.5	51	104	54.9	64	112	79.3	72
78	8.5	38	93	33.0	53	105	57.3	65	113	81.7	73
82	11.0	42	93	35.4	53	105	59.7	65	116	84.1	76
83	13.4	43	94	37.9	54	107	62.1	67	119	86.6	79
83	15.9	43	94	40.3	54	109	64.6	69	121	89.0	81
83	18.3	43	95	42.7	55	110	67.0	70	124	91.5	84
84	20.7	44	99	45.2	59	110	69.5	70	124	93.9	84
85	23.2	45	100	47.5	60	111	71.9	71	124	96.4	84
									129	98.8	89

* Data furnished by the courtesy of Mr. L. Standish Hall.

Fig. 4 shows the curve for the annual discharge through the outlet of Lake Champlain,* plotted from data in Table 6. The curves of this diagram are particularly interesting since the coefficient of variation, as given in Table 1 is negative. The duration curve is flat and concave upward while the frequency curve is high, with the mode greater than the mean and the average range unusually small, only about 22% as against 60% in Fig. 5.

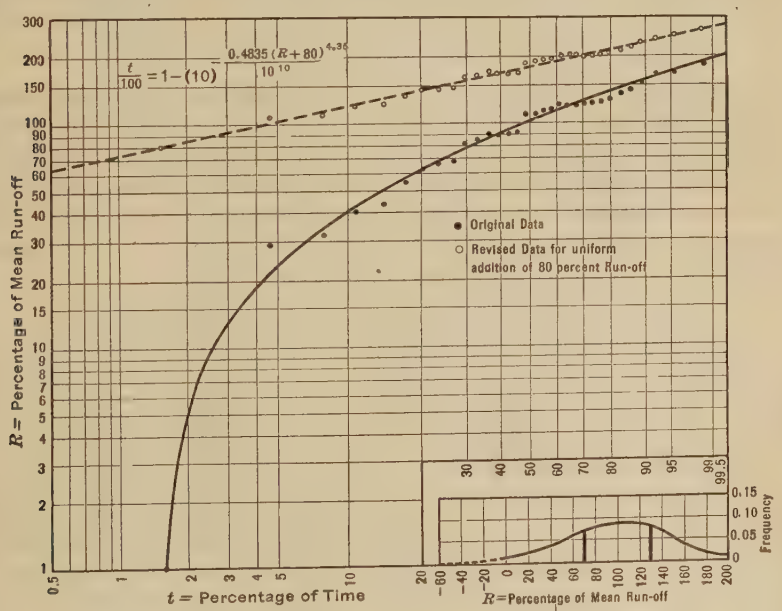
$\frac{t}{100} = 1 - (10)^{-\frac{0.4835\,(R+80)^{4.36}}{10^{10}}}$
Original Data
Revised Data for uniform addition of 80 percent Run-off
R = Percentage of Mean Run-off
t = Percentage of Time
R=Percentage of Mean Run-off
Frequency

Fig. 5.—Curves for Mackenzie River, at Wartook, Australia.

* The writer is indebted to Mr. L. Standish Hall, who kindly furnished the data for the Lake Champlain outlet, the several Australian rivers, and the Sweetwater River, California, all of which have added greatly to the value of this paper by the variety of conditions

Fig. 5 shows a decided difference in the duration curve (plotted from Table 7), which is concave downward. The lower limit is apparently negative and a constant must be added to the values of R to bring the data into suitable form for plotting the auxiliary curve. The frequency curve for this river is comparatively flat with the mode greater than the mean, as in Fig. 4, but with the beginning of the curve at — 80%, shown by the dotted part at the left. These curves should not be interpreted as indicating a possible negative run-off, but approximately zero run-off probably 1½% of the time. Actually a minimum of 1% of the mean was recorded in 32 years' observations. The duration curve was made to pass through this point and averages the remainder of the data remarkably well.

TABLE 7.—Mackenzie River at Wartook, Australia.*

Percentage of mean run-off, R.	Percentage of time, t.	$R + 80$.	Percentage of mean run-off, R.	Percentage of time, t.	$R + 80$.	Percentage of mean run-off, R.	Percentage of time t.	$R + 80$.	Percentage of mean run-off, R.	Percentage of time, t.	$R + 80$.
1	1.6	81	85	32.8	165	121	63.9	201	171	95.3	251
29	4.7	109	90	36.0	170	121	67.2	201	185	98.4	265
32	7.8	112	90	39.1	170	122	70.3	202	...	....	...
41	10.9	121	90	42.2	170	124	73.2	204	...	....	...
44	14.1	124	91	45.3	171	126	76.6	206	...	....	...
55	17.2	135	111	48.5	191	128	79.7	208	...	....	...
63	20.3	143	111	51.6	191	138	82.8	218	...	....	...
67	23.4	147	115	54.7	195	142	85.9	222	...	....	...
68	26.8	148	115	57.8	195	155	89.1	235	...	....	...
82	29.7	162	120	60.9	200	167	92.2	247	...	....	...

* Data furnished by the courtesy of Mr. L. Standish Hall.

The data in Table 8 was condensed by omitting the two lowest records and taking alternate items after arranging them in order of magnitude. This does not materially alter the shape in which the data are plotted (Fig. 6) while it serves, as well as would the complete record, to illustrate the particular type of frequency curve shown. This rather rare type is indicated in the equation of the curve by a value of c of less than unity. The curve begins at the lower limit at infinity and does not have any hump.

TABLE 8.—Murray River at Mildura, Australia.*

Percentage of mean run-off, R.	Percentage of time, t.	$R - 51.5$.	Percentage of mean run-off, R.	Percentage of time, t.	$R - 51.5$.	Percentage of mean run-off, R.	Percentage of time, t.	$R - 51.5$.
52	2.1	0.5	76	43.8	24.5	160	85.4	108.5
53	6.3	1.5	86	47.9	34.5	170	89.6	118.5
54	10.4	2.5	92	52.1	40.5	198	93.7	146.5
55	14.6	3.5	94	56.2	42.5	301	97.9	249.5
56	18.7	4.5	101	60.4	49.5	....	....	.....
59	22.9	7.5	122	64.6	70.5	....	....	.....
62	27.1	10.5	133	68.7	81.5	....	....	.....
63	31.3	11.5	135	72.9	83.5	....	....	.....
68	35.4	16.5	136	77.1	84.5	....	....	.....
72	39.6	20.5	148	81.3	96.5	....	....	.....

* Data condensed from Table 16, *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 246.

Thus far, all the variations in the duration and frequency curves shown can be expressed mathematically by suitable variations in the values and signs of the constants of Equation (*A*). (Equation (*S*) is only a special case of Equation (*A*), in which the constant, *a*, becomes zero.) The limits of these curves are zero and ∞. Of the six curves shown, Figs. 2, 3, and 6 belong to Type III of Pearson's skew frequency curves, while Figs. 1, 4, and 5 belong either to his Type V or Type VI. (The distinction between these last two types is not clear.)

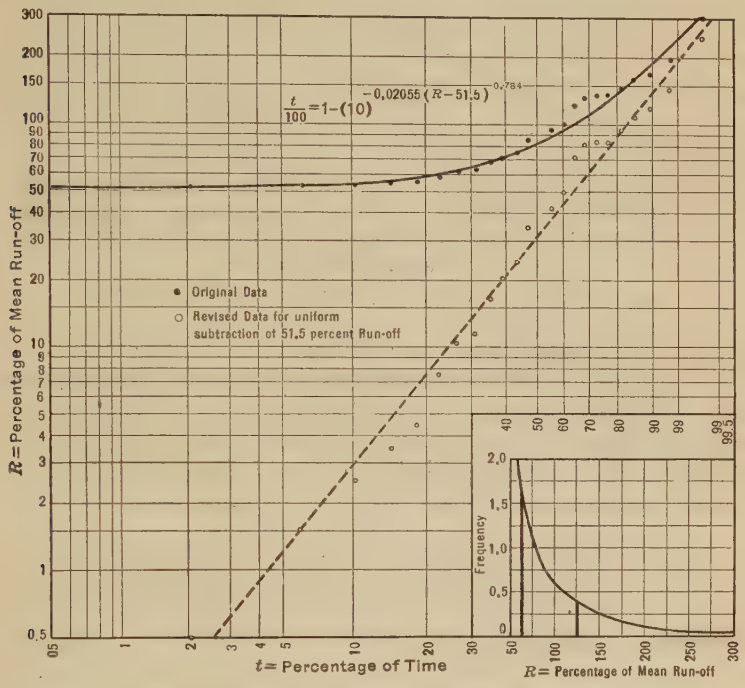
$\frac{t}{100}=1-(10)^{-0.02055\,(R-51.5)^{0.784}}$
Original Data
Revised Data for uniform subtraction of 51.5 percent Run-off
R = Percentage of Mean Run-off
t = Percentage of Time
Frequency
R = Percentage of Mean Run-off

FIG. 6.—CURVES FOR MURRAY RIVER, AUSTRALIA.

The next four diagrams (Figs. 7 to 10) illustrate the graphical and mathematical representation of data requiring slightly different treatment in order to permit the plotting of a linear auxiliary curve. For these curves it is necessary to compute the columns of values of $b - R$ (Tables 9 and 10), as well as the values of $R - a$, as the shape of the curves shows that Equation (*B*) must be used. The required value of the constant, *b*, can be found approximately by trial, so that the ratios of $R - a$ to $b - R$, plotted to the corresponding values of the percentage of time, will approximate a straight line for guidance in drawing the necessary auxiliary curve. The ratios can be computed with sufficient accuracy on an ordinary 10-in. slide-rule. That this method of treating the data is required is indicated when the original data show an evident reversal in the direction of curvature when plotted on skew frequency paper. The constant, *b*, serves to straighten the right-hand end of the data in the same way that the constant, *a*, was used in the first illustrations to straighten the

left-hand part. A little practice will make it possible to select practical values of these constants, in most cases after two or three preliminary trials.

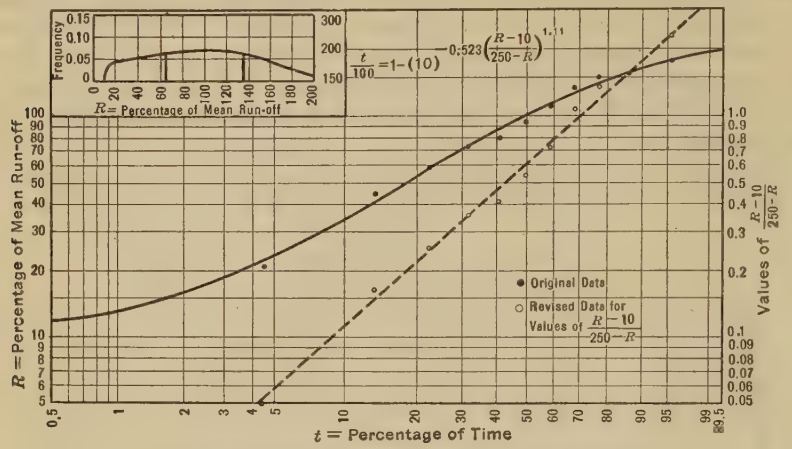
Frequency
R = Percentage of Mean Run-off
$\frac{t}{100} = 1 - (10)^{-0.523\left(\frac{R-10}{250-R}\right)^{1.11}}$
R = Percentage of Mean Run-off
t = Percentage of Time
Values of $\frac{R-10}{250-R}$
Original Data
Revised Data for Values of $\frac{R-10}{250-R}$

FIG. 7.—CURVES FOR DONNER CREEK AT TRUCKEE, CALIF.

Fig. 7 shows the curve for the data of Donner Creek, California, given in Table 9. While the records are too few to give a smooth curve they serve to illustrate the method of computing and plotting a linear function for data following skew frequency laws in which both limits are evidently finite. The frequency curve shows a flat top, indicating nearly equal probability for the entire range of average conditions.

TABLE 9.—DONNER CREEK AT TRUCKEE, CALIFORNIA.*

Percentage of mean run-off, R. (1)	Percentage of time, t. (2)	$R - 10$. (3)	$250 - R$. (4)	$\frac{R-10}{250-R}$ (5)
21.5	4.5	11.5	228.5	0.050
44.4	13.6	34.4	205.6	0.167
58.8	22.7	48.8	191.2	0.256
72.9	31.8	62.9	177.1	0.355
79.3	40.9	69.3	170.7	0.406
94.2	50.0	84.3	155.8	0.54
110.3	59.1	100.3	139.7	0.72
134.4	68.2	124.4	115.6	1.08
149.2	77.3	139.2	100.8	1.36
156.3	86.4	146.3	93.7	1.56
178.0	95.5	168.0	72.0	2.33

* Data from Table 1, *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 196.

The curve for West Canada Creek (Fig. 8), from the data of Table 10, seems to be a common type for hydraulic data. The total range between limits is quite small on account of the relatively high value of the minimum. This frequency curve also has a nearly flat top across the average range, but begins at infinity at the left. This merely means that it is the best mathematical

expression that will fit the case and not that there is an infinite probability of the occurrence of the minimum limiting run-off. If the selection of the limits has been properly made it is highly improbable that actually either one would ever be reached. It should be noted that the value of the exponent, c, in the equation for this curve is less than unity, while for Donner Creek it is greater

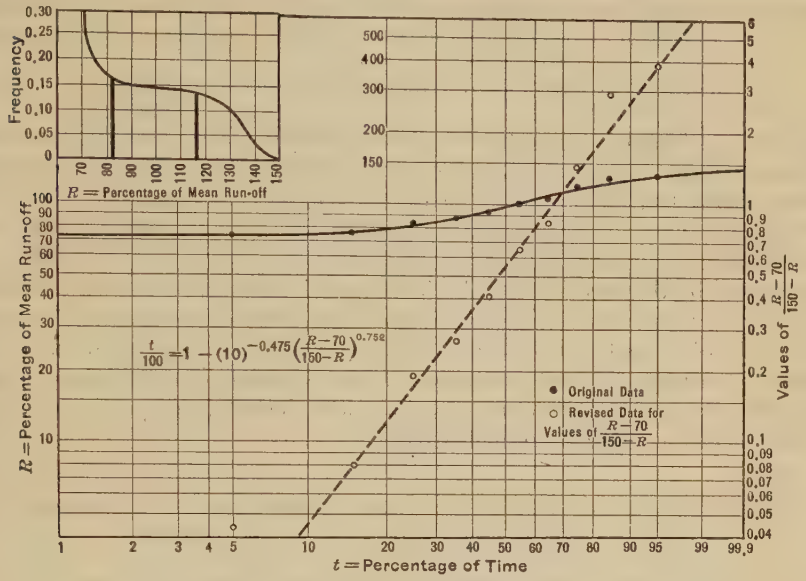
Frequency
R = Percentage of Mean Run-off
R = Percentage of Mean Run-off
t = Percentage of Time
Values of (R − 70)/(150 − R)
$\frac{t}{100} = 1 - (10)^{-0.475\left(\frac{R-70}{150-R}\right)^{0.752}}$
Original Data
Revised Data for Values of (R − 70)/(150 − R)

FIG. 8.—CURVES FOR WEST CANADA CREEK AT TWIN ROCK BRIDGE.

than unity. Both these curves are expressed mathematically by the use of Equation (B) and belong to Type I of the Pearson curves. Fully one-third of the curves plotted by the writer are of this type. Different values of the exponent, c, indicate the same characteristics as to the beginning of the left-hand end of the frequency curve for Equation (B), as for Equation (A). However, the value of the exponent for zero skewness could not be determined for Equation (B).

TABLE 10.—WEST CANADA CREEK AT TWIN ROCK BRIDGE, NEW YORK.*

Percentage of mean run-off, R. (1)	Percentage of time, t. (2)	$R - 70$. (3)	$150 - R$. (4)	$\frac{R-70}{150-R}$. (5)
73.4	5	3.4	77.6	0.044
75.9	15	5.9	74.1	0.080
83.0	25	13.0	67.0	0.194
86.9	35	16.0	63.1	0.268
93.4	45	23.4	56.6	0.413
101.4	55	31.8	48.6	0.646
106.3	65	36.3	43.7	0.830
117.5	75	47.5	32.5	1.461
129.6	85	59.6	21.4	2.922
132.6	95	62.6	17.4	3.820

* Data from Table 5, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 171.

The data for the Sweetwater River at the Sweetwater Dam, California, given in Table 11, present a most interesting and unusual case. Of the thirty-two records given, five show zero run-off for the year. On the other hand the maximum is nearly ten times the mean. When these data are plotted on skew frequency paper (Fig. 9) they form a reverse curve, but the direction of curvature is opposite to that shown by Figs. 7 and 8. However, it is only necessary to change the minus signs to plus in the function, $\frac{R - a}{b - R'}$, in order to draw an auxiliary curve by computing this linear function in the same manner as for Figs. 7 and 8. The duration curve and the approximate frequency curve for the Sweetwater River (Fig. 9), show that during about 18% of the years, no run-off may be expected at the dam. In this case the frequency curve has an apparent origin at — 50% of the time, as indicated by the dotted part. The upper limit of this curve becomes ∞ by the change in the sign to $b + R$. The excessive skewness causes the range of average run-off to extend from about 9.5 to 110% of the mean. Equation (B), as herein used, should make possible a scientific investigation of the probable frequency of the occurrence of such conditions which may be produced by varying combinations of supply and demand, and it is hoped this method will prove useful for such studies.

TABLE 11.—Sweetwater River at the Sweetwater Dam, California.*

Percentage of mean run-off, R.	Percentage of time, t.	$R + 50$.	$100 + R$.	$\frac{R + 50}{100 + R}$.	Percentage of mean run-off, R.	Percentage of time, t.	$R + 50$.	$100 + R$.	$\frac{R + 50}{100 + R}$.
0	1.6	50	100	0.500	38	48.4	88	138	0.638
0	4.7	50	100	0.500	42	51.6	92	142	0.648
0	7.8	50	100	0.500	43	54.7	93	143	0.650
0	10.9	50	100	0.500	59	57.8	109	159	0.685
0	14.1	50	100	0.500	63	60.9	113	163	0.693
2	17.2	52	102	0.510	85	64.1	135	185	0.730
5	20.3	55	105	0.523	94	67.2	144	194	0.741
6	23.4	56	106	0.528	99	70.3	149	199	0.748
8	26.6	58	108	0.537	100	73.4	150	200	0.750
8	29.7	58	108	0.537	126	76.6	176	226	0.778
19	32.8	69	119	0.580	123	79.7	183	233	0.785
22	35.9	72	122	0.590	156	82.8	206	256	0.804
25	39.1	75	125	0.600	167	85.9	217	267	0.812
26	42.2	76	126	0.603	185	89.1	235	285	0.825
31	45.3	81	131	0.618	216	92.2	266	316	0.841
..	....	..	...	.....	452	95.3	502	552	0.910
..	....	..	...	.....	990	98.4	1 040	1 990	0.954

* These data were supplied by the courtesy of Mr. L. Standish Hall.

Special Cases in Hydraulics and Economics.—If the lower limit for the data in Fig. 9 had been positive instead of negative, the curve would have been concave upward at the left with the same strong upward tendency at the right. No hydraulic data have been found to illustrate this condition, but Fig. 10, plotted from data on the valuation of house property in England

and Wales, by Professor Pearson (Table 12), is a good example. Both the integral and differential curves (Fig. 10) have the same general appearance as in Fig. 6, but with much sharper curvature. The data cannot be closely approximated by Equation (*A*).

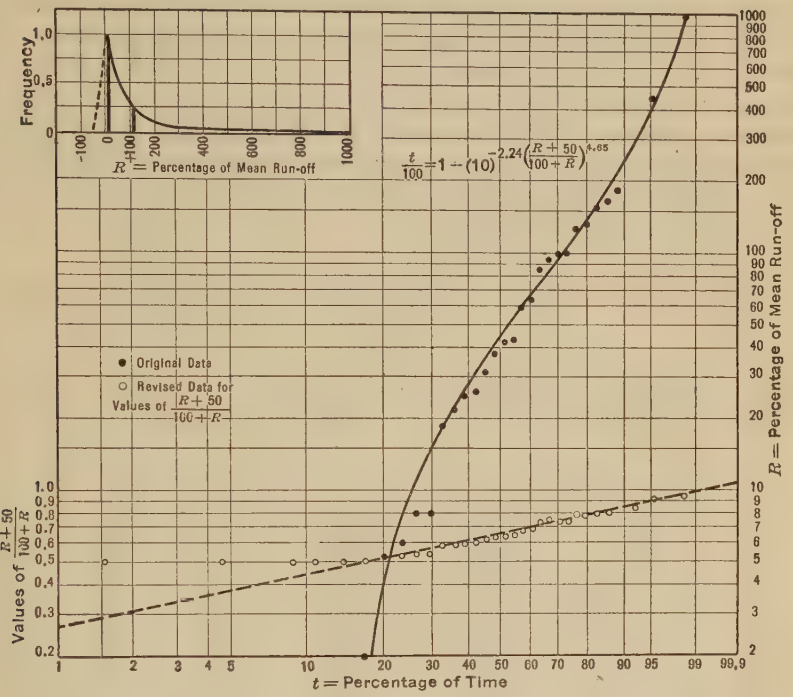

FIG. 9.—CURVES FOR SWEETWATER RIVER AT SWEETWATER DAM, CALIFORNIA.

The last diagram of this series (Fig. 11) illustrates another unusual case. Repeated trials, using numerous different combinations of limits, failed to produce even an approximately satisfactory curve by the process shown in the preceding diagrams. Finally, following a hint in the suggestive papers on skew frequency curves by Dr. Kapteyn (to be referred to later), it was found that by plotting the data (Table 13) from right to left instead of from left to right as in every other instance, a theoretical curve could be drawn which would follow closely four-fifths of the plotted points. In Fig. 11 the values of t are given at the top and increase from right to left while at the bottom those which increase from left to right represent $100\% - t$. It is evident that the two series are symmetrical about the ordinate for 50 per cent. This change amounts to substituting $n - t$ for the left-hand member of the fundamental equations, and the constant term, n, is then eliminated. Fig. 11 shows that the slope of the auxiliary curve is negative and hence the sign of the exponent, c, is negative. While for about one-fifth the data at the upper end the curve does not follow the plotted points closely, it does indicate the same general

tendency as to direction.* Evidently there is some cause or set of causes producing this deviation in the records, which cannot be expressed by such a simple law as that which seems to fit the remainder of the data. The lower limit of the frequency curve is apparently at — 10 per cent. Hence zero run-off may be expected. By substituting zero for R, $f(R)$ reduces to 0.025, and it is found that t would then be 0.2%, or about once in 500 years there might be a season of no run-off. While the maximum may be perhaps 300% of the mean run-off, years of less than average discharge will be much more frequent. In fact, one-half the time the total annual discharge of this river will probably be less than 65% of the mean. Attention is called to this point in order to show the danger of basing engineering designs on information as to the mean only and the valuable information sometimes obtained from such frequency studies as those illustrated.

TABLE 12.—Valuation of House Property in England and Wales, 1885-1886.*

Value, in £ sterling, x. (1)	Number of houses, in thousands. (2)	Observed frequency. (3)	Percentage less than value in Column (1). (4)	$x - 4$. (5)	$160 + x$. (6)	$\frac{x-4}{160+x}$. (7)
0	.....	.....	.....	...	.....	.....
10	3 175	0.5446	54.46	6	170	0.035
20	1 451	0.2489	79.35	16	180	0.089
30	441.6	0.0758	86.93	26	190	0.137
40	259.8	0.0446	91.39	36	200	0.180
50	151.0	0.0259	93.98	46	210	0.219
60	90.4	0.0155	95.53	56	220	0.255
80	104.1	0.0179	97.32	76	240	0.317
100	47.3	0·0081	98.13	96	260	0.369
150	58.9	0.0101	99.04	146	310	0.471
300	38.0	0.0065	99.69	296	460	0.643
500	8.8	0.0015	99.84	496	660	0.752
1 000	3.0	0.0005	99.89	996	1 160	0.858
1 500	1.0	0.0002	100.01	...	.....	.....

* Original data from *Philosophical Transactions*, Vol. 186, p. 396; reprinted in Kapteyn's "Skew Frequency Curves" (First Paper), p. 35.

The equations of the duration curves for all the data illustrated are given on the diagrams. Equations for the frequency curves can be obtained by the ordinary process of differentiation. The general forms for these second equations have already been given, being such that the constants can be readily computed from those of the duration curve and the equations can also be easily formed without using the actual process of differentiation.

The form of skew frequency blank diagram given in Fig. 12 is identical with those used in all the foregoing examples. An engineer wishing to try out this method or experiment with his own data will find this blank convenient for his use. A trial will demonstrate the simplicity of the principles involved and, it is hoped, the discussions here given will enable any one successfully to draw skew frequency curves fitting his own case.

* It was afterward found that much closer agreement with the data at the upper end of the curve of Fig. 11 could be obtained by using the most general form of Equation (B), involving five constants. (See dotted curve and its equation, Fig. 11.)

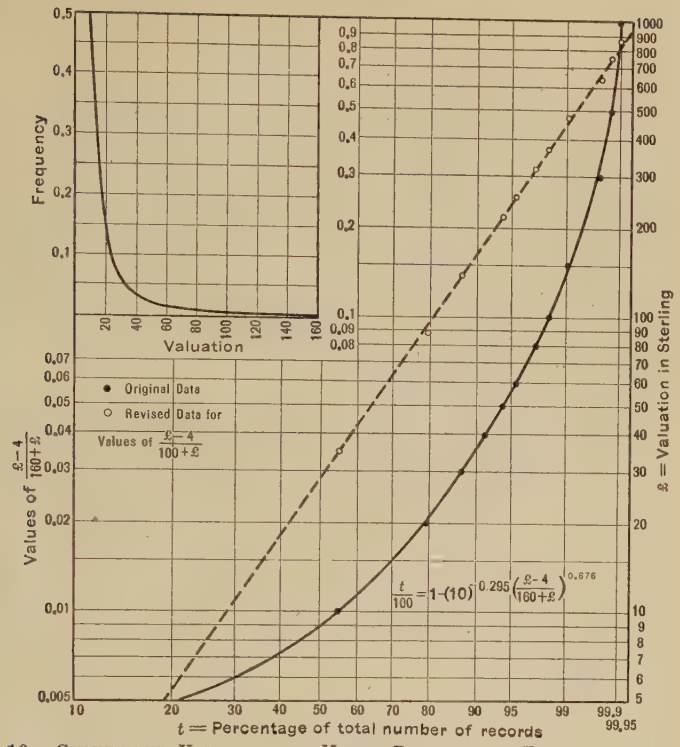

FIG. 10.—CURVES FOR VALUATION OF HOUSE PROPERTY IN ENGLAND AND WALES, 1885 TO 1886.

TABLE 13.—WIMMERA RIVER AT GLENORCHY, AUSTRALIA.*

Percentage of mean run-off, R.	Percentage of time, t.	$100-t$.	$R+10$.	$400-R$.	$\frac{R+10}{400-R}$	Percentage of mean run-off, R.	Percentage of time, t.	$100-t$.	$R+10$.	$400-R$.	$\frac{R+10}{400-R}$
5	1.7	98.3	15	395	0.038	65	51.7	48.3	75	335	0.224
10	5.0	95.0	20	390	0.051	75	55.0	45.0	85	325	0.262
16	8.3	91.7	26	384	0.068	85	58.3	41.7	95	315	0.302
19	11.7	88.3	29	381	0.076	86	61.7	38.3	96	314	0.306
20	15.0	85.0	30	380	0.079	88	65.0	35.0	98	312	0.314
21	18.3	81.7	31	379	0.082	99	68.3	31.7	109	301	0.362
28	21.7	78.3	38	372	0.102	159	71.7	28.3	169	241	0.702
31	25.0	75.0	41	369	0.111	164	75.0	25.0	174	236	0.737
31	28.3	71.7	41	369	0.111	215	78.3	21.7	225	185	1.216
36	31.7	68.3	46	364	0.126	216	81.7	18.3	226	184	1.228
36	35.0	65.0	46	364	0.126	238	85.0	15.0	248	162	1.531
39	38.3	61.7	49	361	0.136	245	88.3	11.7	255	155	1.645
49	41.7	58.3	59	351	0.168	262	91.7	8.3	272	138	1.970
54	45.0	55.0	64	346	0.185	274	95.0	5.0	284	126	2.255
59	48.3	51.7	69	341	0.202	278	98.3	1.7	288	122	2.360

* These data were furnished by the courtesy of Mr. L. Standish Hall.

Extension of Short-Term Record Series.—In the practical use of the foregoing method of plotting skew frequency data, the best results will be obtained if the series contain at least twelve or fifteen records. Still longer series are likely to give smoother curves, which can be extended with greater confidence. Some students have objected to this method of analysis on the ground that, if a series of, say, five or six consecutive records are taken from a longer series and treated as if they constituted the entire series, the curves and equations might be quite different from those obtained if all the observations were used. The same objection would apply to any other method of investigation if only a small number of observations are taken into consideration. The fluctuation in 5 and 10-year averages computed from long-time records of rainfall is well recognized, and attention has frequently been called to the danger of basing conclusions on such short-time means.

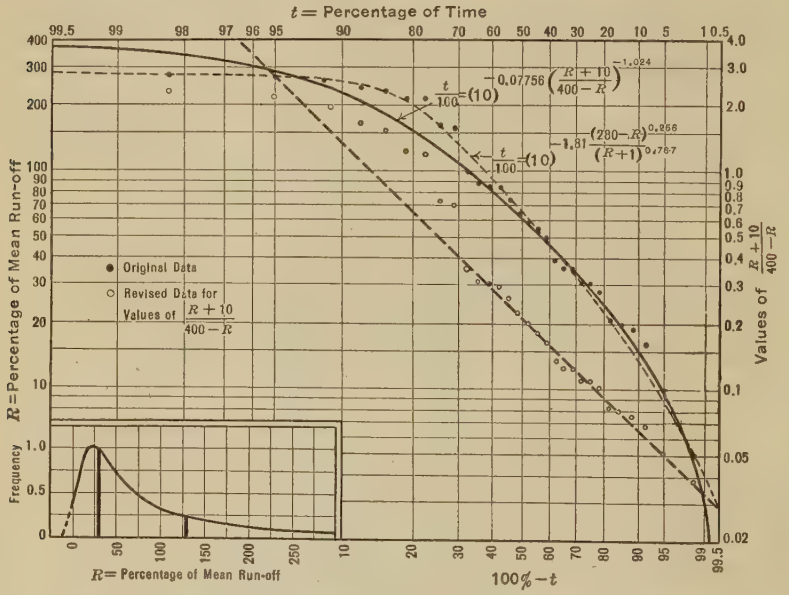

FIG. 11.—CURVES FOR WIMMERA RIVER AT GLENORCHY, AUSTRALIA.

It is possible to avoid this difficulty to some extent even when there are only from five to ten records available if there is a much longer record at another point on the same or a neighboring stream, provided the difference in conditions is not too great. A method similar to that indicated* by Joel B. Cox, Assoc. M. Am. Soc. C. E., has also been used by other engineers in dealing with short-time records. Its application to the foregoing method of treating hydraulic data is given here in order to make this paper a complete reference work for those who may not be familiar with its use.

In Table 14 are given run-off data for the Carson River, at Empire, Nev., and for the West Fork of the Carson River at Woodfords, Calif. Columns

* "Periodic Fluctuations in Rainfall in Hawaii," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 461.

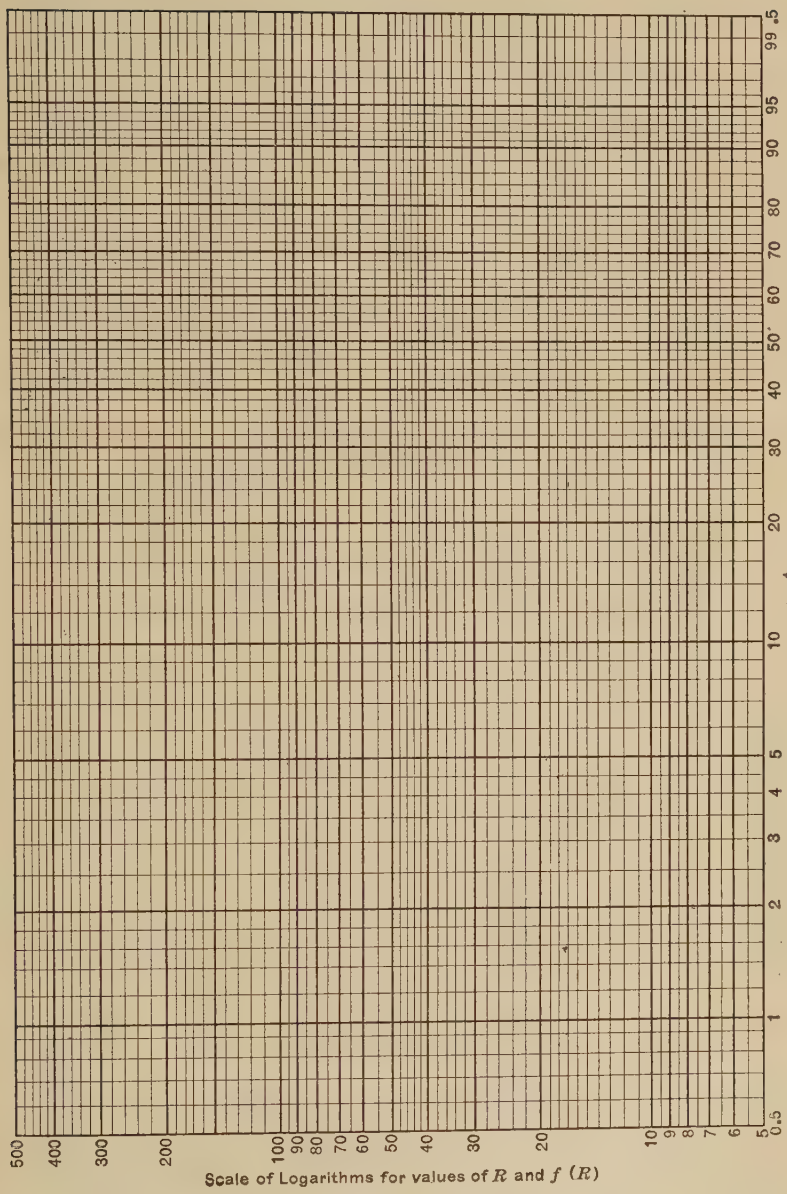

FIG. 12.—SPECIALLY RULED CHART FOR PLOTTING SKEW FREQUENCY DATA.

(1), (2), and (3) are taken directly from Mr. Hall's paper previously mentioned. In Column (4) the data of Column (2) for the Carson River are rearranged in the order of volume of total annual run-off. In Column (5) appears the percentage of the total time for which the corresponding percentages of the mean annual run-off will not be exceeded, on the average. In Column (6) is the cologarithm of the decimal fraction $\left(1 - \frac{t}{100}\right)$. It should be noted that all the quantities in Columns (4), (5), and (6) increase in the same direction. Now, suppose that only the last five records given in Column (3) for the West Fork are available and it is proposed to investigate the probable frequency of high and low discharges at Woodfords, Calif. These five records are listed in Column (7) opposite the record for the Carson River at Empire for the same year, as it appears in Column (4). If skew frequency paper is not available, ordinary logarithmic paper may be used, as illustrated in Fig. 13. On such paper the percentage of the mean run-off taken from Column (7) is plotted to the corresponding cologarithm given in Column (6).

TABLE 14.—Annual Run-off of Carson River and West Fork of Carson River, at Woodfords, California.*

Seasonal year.	Carson River, at Empire, Nev.	West Fork of Carson River, Woodfords, Calif., R.	Carson River, in order of volume.	Percentage time record is not exceeded, t.	colog $\left(1 - \frac{t}{100}\right)$.	West Fork of Carson River, 1910-11 to 1914-15, R.	Percentage of time for five records, t.	colog $\left(1 - \frac{t}{100}\right)$.	$R - 64$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1900-01	96.3	92.9	44.8	2.95	0.013	65.1	10	0.046	1.1
1901-02	65.9	88.4	48.0	8.8	0.040	66.0	30	0.155	2.0
1902-03	76.4	75.8	51.2	14.7	0.069	....	..	.....	....
1903-04	124.6	113.3	62.6	20.6	0.100	....	..	.....	....
1904-05	62.6	73.2	65.9	26.5	0.134	....	..	.....	....
1905-06	134.9	146.4	76.4	32.4	0.170	....	..	.....	....
1906-07	182.3	187.4	80.5	38.2	0.209	77.6	50	0.301	13.6
1907-08	51.2	64.6	88.6	44.1	0.253	....	..	.....	....
1908-09	108.3	125.8	96.3	50.0	0.301	....	..	.....	....
1909-10	88.6	92.0	108.3	55.9	0.356	....	..	.....	....
1910-11	155.6	133.9†	117.3	61.8	0.418	....	..	.....	....
1911-12	44.8	65.1†	117.6	67.6	0.489	....	..	.....	....
1912-13	48.0	66.0†	124.6	73.5	0.577	....	..	.....	....
1913-14	145.1	96.5†	134.9	79.4	0.686	....	..	.....	....
1914-15	80.5	77.6†	145.1	85.3	0.833	96.5	70	0.523	32.5
1915-16	117.6	.....	155.6	91.2	1.056	133.9	90	1.000	69.9
1916-17	117.3	.....	182.3	97.05	1.530	....	..	.....	....

* Data from Table 1, *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 196.
† Data distributed according to Carson River frequency numbers.

The points are shown by the solid dots in the diagram. A smooth curve can evidently be drawn through four of these points. In order to plot this theoretical curve it is first necessary to plot the auxiliary curve. It is found by trial that if 64 is subtracted from each quantity in Column (7), the first, third, and last points will fall on a straight line when the quantities, $R - 64$, in Column (10) are plotted to the corresponding cologarithms from Column (6). This is shown by the dotted straight line and the crossed circles in Fig. 13,

from which the dotted curve was then plotted. The curve shown by the solid line was plotted in the ordinary manner, using the full record of fifteen years at Woodfords, treated without reference to the records at Empire. The complete data are shown by the open circles in Fig. 13, but the auxiliary curve is not shown. A closer agreement between the two curves might have been obtained if the records for the first five years or more had been used in place of only the five last years. If the five records used had been treated without reference to the longer record on the same stream, the data given in Columns (8) and (9) of Table 14 would have been computed and plotted as shown by the small triangles.

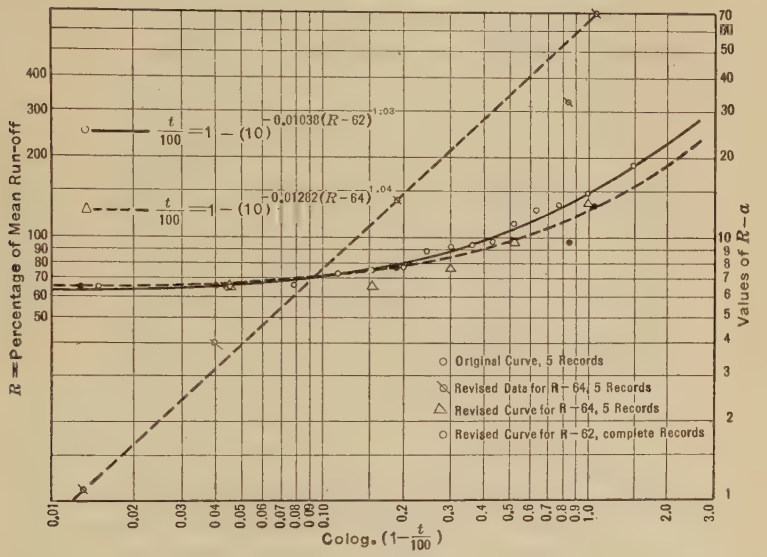
R = Percentage of Mean Run-off
Values of R − a
Colog. (1 − t/100)
○ Original Curve, 5 Records
Revised Data for R − 64, 5 Records
Revised Curve for R − 64, 5 Records
Revised Curve for R − 62, complete Records

FIG. 13.—CURVES FOR WEST FORK OF CARSON RIVER, AT WOODFORDS, CALIF.

A curve through these points would depart widely from the theoretical curve for the whole series throughout most of its length, but would cross it near each end, while the curve obtained by the aid of the long series departs considerably only at one end. It is evident that records for more than five years are usually necessary for reliable results. The equations of any of these curves can be obtained most easily, after finding the approximate limits of the curves, by substituting in Equations (S_2), (A_2), or (B_2), as the case may be, the co-ordinates of two well-defined points on any curve. All the numerical equations have been computed in this manner.

Population Data.—Another line of investigation in which these equations should prove useful is in the representation and prediction of the growth of municipal and general population.* Their application to such studies is shown in Figs. 14, 15, and 16. These diagrams also illustrate the straight line plotting of the logarithmic equations, (A_2) and (B_2), on ordinary co-ordinate paper.

* Experience has shown that population growth seems to follow a law best represented by an ogee curve, just as has been found to be the case with hydraulic data.

In Table 15, the probable maximum population of East Orange, N. J., is taken as 125 000. This is the value to be assigned to n in the equations, here represented by P. By trial it is found that if the origin is taken as the year 1740 ($= a$), the data are closely approximated. In order to make the significance of the various terms in the equation clearer, the expression, (year — 1740), is replaced by x, and p, representing the population of any year, is used instead of t. The quantities in Table 15, Columns (2) and (3), are computed from Column (1), and Columns (5), (6), and (7) are computed with $P = 125\,000$ and the quantities in Column (4). Corresponding values are plotted from Columns (3) and (7) as shown by the circles in Fig. 14.

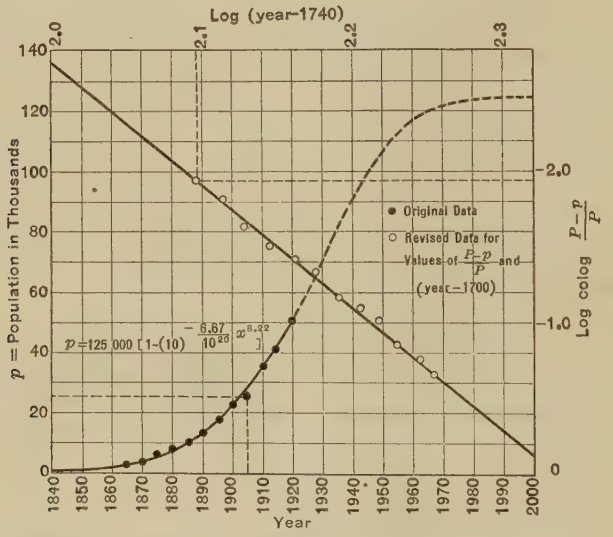

FIG. 14.—CURVES FOR POPULATION OF EAST ORANGE, N. J.

The auxiliary curve can then be drawn and fits the data quite well. The scale of log colog $\left(\frac{P-p}{P}\right)$ was reversed from the usual direction so that the auxiliary curve would not parallel the general direction of the final curve, hence the slope of the auxiliary curve is not negative as might be thought.

By substituting the co-ordinates of the first and last points on the auxiliary curve in Equation (A_2) the values of the remaining constants are found and the following approximate equation can be written:

$$p = 125\,000\left[1 - (10)^{\frac{-6.67}{10^{20}}(y - 1740)^{8.22}}\right]$$

The reverse curve shown in Fig. 14 can now be plotted from this equation or graphically by readings taken from the auxiliary curve. The value of the exponent, c, in this equation is large and hence the accuracy of the plotting may be increased by using different lengths for the units of the scales for log x and log colog $\frac{P-p}{P}$.

This type of curve is easily plotted if a reasonable estimate can be made for the probable maximum total population that can be supported by the area under consideration. An approximate method to assist in arriving at this estimate will be given in connection with the next illustrative example. It is to be noted that the reverse curve follows the economic laws, now generally recognized, of an increasing rate of growth up to a certain point followed by a gradually decreasing rate.

TABLE 15.—Population of East Orange, New Jersey.*
(Assumed maximum population, P, = 125 000.)

Year.	Year—1740, x.	log x.	Population $= p$.	$\frac{P-p}{P}$	colog. $\left(\frac{P-p}{P}\right)$	log colog. $\left(\frac{P-p}{P}\right)$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1865	125	2.097	3 206	0.974	0.0114	−1.943
1870	130	2.114	4 315	0.965	0.0155	−1.810
1875	135	2.130	6 497	0.947	0.0237	−1.625
1880	140	2.146	8 349	0.933	0.0301	−1.521
1885	145	2.161	10 328	0.917	0.0376	−1.425
1890	150	2.176	13 282	0.893	0.0491	−1.309
1895	155	2.190	17 927	0.856	0.0675	−1.171
1900	160	2.204	21 506	0.828	0.0820	−1.086
1905	165	2.218	25 175	0.799	0.0975	−1.011
1910	170	2.230	34 731	0.722	0.1415	−0.849
1915	175	2.243	40 969	0.672	0.1726	−0.763
1920	180	2.255	50 712	0.594	0.2262	−0.646

* Data from Fig. 17, p. 22, of "City Plan for East Orange, N. J." report of City Plan Commission, 1922.

The growth of population in the United States as a whole affords another excellent example for the application of this equation. The statistics are so generally available that the tabulation is omitted. It would be made in exactly the same manner as for the preceding case. The dotted line in Fig. 15 is an auxiliary curve based on the assumption that the maximum population which can be supported by the area of the United States is 400 000 000 and that the curve begins with the year 1620.

This line fits the data quite well along the central part but there is a slight departure in the points as plotted to the left of the middle, which can be reduced by taking an earlier year of origin. The deviation of the direction indicated by the last four points at the right from the direction of the dotted line may be due to an over-estimate of the value of the limit. The equation for the population curve as obtained from this auxiliary curve is:

$$p = 400\left[1 - (10)^{\frac{-5174.4\,(y-1620)^{6.28}}{10^{20}}}\right]$$

in which, p is the population, in millions, and y is the corresponding year.

With the values of the constants of this equation substituted in the formula for the mode it is found that, with a limit of 400 000 000 for the total population, the probable year of maximum rate of growth would be 1970. Examination of the statistics shows that the average rate of increase was

greatest for the 10-year period from 1900 to 1910. There was a marked decrease of rate during the next 10-year period which was doubtless due, at least in part, to the practical suspension of immigration and other economic results caused by the World War. While the decrease in rate is, therefore, probably more or less temporary, it will be assumed for the purposes of illustration that the year of maximum rate of increase was 1914, that the population for that year, as found by interpolation, was 97 500 000 and that the average annual increase of 1 600 000 during the previous decade also applied for that year. It was found by trial that with the year of origin kept

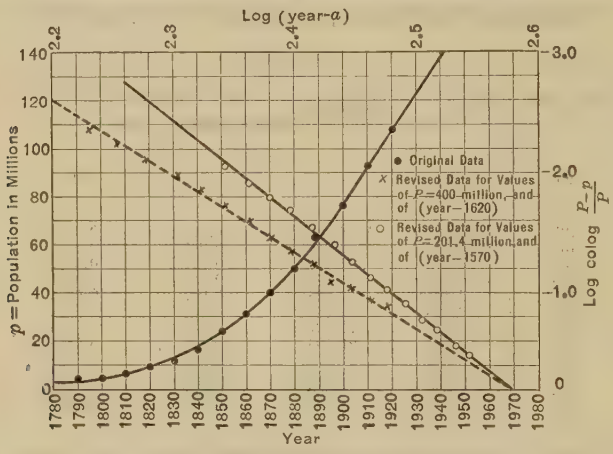

FIG. 15.—CURVES FOR POPULATION OF UNITED STATES.

constant, a large change in the value of P would cause a relatively small change in the value of c. Now assume that a and c remain constant and are found by trial by using any reasonable value of P; if the data show that the probable year of maximum rate of increase has been passed, and if the values of p and p' (population and annual increase for the given year) are computed for that year, then an approximate value for P can be obtained as follows:

$$p = P - P\,e^{-k\,(y-a)^c}$$

$$\text{Mode} = (y_{\text{max.}} - a) = \left(\frac{c-1}{c\,k}\right)^c$$

$$P = \frac{p'}{c\,k\,(y-a)^{c-1}\,e^{-k\,(y-a)^c}}$$

$$p' = c\,k\,P\,(y-a)^{c-1}\,e^{-k\,(y-a)^c}$$

$$(y_{\text{max.}} - a)^c = \frac{c-1}{c\,k}$$

Then,

$$p = P - \frac{p'}{c\,k\,(y-a)^{c-1}} = P - \frac{p'}{c\,k\left(\frac{c-1}{c\,k}\right)(y-a)}$$

Hence, substituting $y_{\text{max.}}$ for y, $p_{\text{max.}}$ for p, and $p_{\text{max.}}'$ for p', the resulting formula is:

$$P = p_{\text{max.}} + \frac{p_{\text{max.}}'\,(y_{\text{max}} - a)}{(c-1)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which, $y_{max.}$ is the year of maximum increase, while $p_{max.}$ and $p_{max.}'$ are the corresponding population and rate of increase.

Substituting in Equation (10) the values proposed for the constants, but with 1570 instead of 1620, the value of P is found to be 201 400 000 for the limiting population. With these values for a and P points were computed and the second auxiliary curve shown by the solid line was drawn. This line fits the data perhaps a little more closely than the first one, although there is little to choose between them. The curve drawn through the original data as shown by the dots in Fig. 15, is plotted from the second auxiliary curve and is really a good fit. Its equation is:

$$p = 200\left[1 - (10)^{\frac{-0.181\ (y\ -\ 1570)^{7.96}}{10^{20}}}\right]$$

Other authors have suggested* the desirability of an equation giving an unsymmetrical curve on account of a possible (if not probable) skewness in the curve for the rate of growth of the United States. The equations herein presented are believed to offer the means for providing any required skewness in the rate curve and to give an integral curve of comparatively simple and usable form. The numerical equation for p is given only for the purpose of illustration and should not be considered as the writer's proposal for basing estimates as to the future population. The value of any such equation depends on the reliability with which the upper limit of population can be estimated and this is more a problem in economics than in engineering. For the United States it is perhaps more probable that the limit is 400 000 000 than 200 000 000.

A population curve somewhat more difficult to represent mathematically is given in Fig. 16. It shows that the growth of New Bedford, Mass., may be considered as the result of the simultaneous operation of two distinct laws depending on entirely different and independent sets of causes. The heavy line drawn to average the plotted points can be resolved into two curves. The first is shown by the light solid line, the "curve of" normal growth. This curve and its equation are of the same form as those shown on Figs. 14 and 15. The dotted curve represents approximately the rise and fall of the population engaged in the whaling industry. This business had such a marked effect on the life of the city that it may be considered independently, as governed by distinct laws. An equation can be formed for each of these secondary curves. That for the total population is then written by adding the right-hand members of the first two equations. If the total population of any year is represented by t and the corresponding year by y, then the equation of the final curve is:

$$t = 170 + 154\ (10)^{\frac{-0.004735\ (y\ -\ 1660)^{18.3}}{10^{40}}} - 16\ (10)^{-0.4865\left[\frac{y\ -\ 1770}{(1855\ -\ y)^2}\right]^{0.488}}$$

The equation of the dotted curve is interesting inasmuch as it is the general form of Equation (G) with the exponent, d, equal to $2c$. While the upper limit of the year in Equation (G) is ∞, the corresponding value of t is not n but passes through the maximum value. n, when $y = b$. and then

* *Engineering News-Record*, December 1, 1921, p. 889.

decreases toward the limit zero. Evidently the whaling industry must have reached its zenith at New Bedford about 1855 and has been declining ever since.

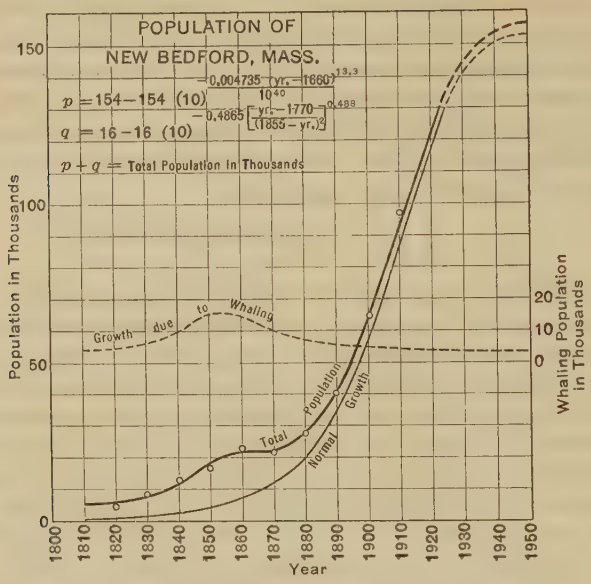

FIG. 16.

Figs. 14, 15, and 16 illustrate possibilities as to the use of the equations in studies for extensions of municipal water supplies and sewage works, and for the future needs of transportation and communication. It is hoped that, with the suggestions made by Kapteyn and Van Uven as to the significance of the various terms in the right-hand member of the equations, it will be possible to arrive at a surer knowledge of the causes producing changes in population and to estimate more accurately their effects.

The following possible application is suggestive only, and is made subject to revision after further study and discussion. The population in any district evidently tends to increase with time, due to births and the arrival of new families. It also depends on the date of origin, the effect of which is constant; an increase in the numerical value of this date tends to decrease the results of the causes of increase of population.* Likewise removals caused by death or otherwise tend to decrease the population, such removals increasing more or less in proportion to the time. If the relation between the causes of increase and decrease in population is fixed and constant, Equation (*A*) may be used to express the law of their resultant effect. If, however, this relation is not a fixed one, Equation (*B*) is required, since it makes it possible to show the result of some fixed cause affecting the tendencies to decrease population, which cause may depend on some fixed date. If the causes that tend toward a reduction in population are such that their total result will

* To express the same idea in non-mathematical terms—the assumption of a later date results in an attendant relative decrease in population.

at some time become greater than the result of the causes tending toward an increase, then Equation (*G*) as modified for the case of New Bedford may be used. One or more of these equations may be used to show the combined result of a great variety of causes affecting phenomena following skew frequency laws.

Biological Application.—A further test of the writer's method was made by fitting a curve to the biological data given in Table 16. The resulting curve differed from any previously investigated, requiring Equation (*C*) for its representation.

The data consist of measurements of the lengths of stalks of flax grown under special conditions. While the conditions were such as to produce an unusual skew distribution of the quantities measured, Dr. Kapteyn, from his analysis of the data, was able to state the general nature and effect of the causes producing the results observed. Similar conclusions can also be drawn from the curves and equations shown on Fig. 17, resulting from the writer's methods. It is stated by Dr. Kapteyn that no figure could be given (perhaps from lack of time, as the work was then in the press) and no equations are given. On the other hand the methods here given permit not only the construction of the duration and frequency curves but also the formation of their equations.

TABLE 16.—Lengths of Stalks of Flax.*

Length of stalks, x.	Number of stalks less than length in Column (1).	Percentage of total, $\frac{y}{100}$.	$220 - x$.	$\frac{1}{220 - x}$.	Length of stalks, x.	Number of stalks less than length in Column (1).	Percentage of total, $\frac{y}{100}$.	$220 - x$.	$\frac{1}{220 - x}$.
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
10	163	12.2	210	0.00476	120	567	42.4	100	0.0100
20	180	13.5	200	0.00500	130	669	50.0	90	0.0111
30	197	14.7	190	0.00526	140	797	59.6	80	0.0125
40	215	16.1	180	0.00556	150	956	71.5	70	0.0143
50	239	17.9	170	0.00588	160	1 094	81.8	60	0.0167
60	265	19.8	160	0.00625	170	1 208	90.3	50	0.0200
70	302	22.6	150	0.00667	180	1 280	95.6	40	0.0250
80	343	25.6	140	0.00715	190	1 320	98.7	30	0.0333
90	383	28.6	130	0.0077	200	1 333	99.6	20	0.0500
100	442	33.0	120	0.0083	210	1 336	99.8	10	0.1000
110	490	36.6	110	0.0091	...	.....	....	..	.....

* Data taken from Appendix to Chapter III, Kapteyn's "Skew Frequency Curves" (Second Paper), p. 69.

The interpretation of the results observed, can best be given in the words of the author:

"There seems to be an almost insuperable impediment against growth. Those individuals, however, who succeed in overcoming the first difficulty then begin to grow very rapidly indeed, the rapidity increasing till the size 25 is reached. After that the growth begins to diminish; it gradually de-

creases, to only half of the maximum growth for the individuals of the size 100 and below one-tenth of the maximum growth for the individuals of size 170. * * *

"All this proves to be in good agreement with what has been really observed. Dr. Tammes writes: 'The case I sent you is as follows: the quantities communicated are stalk-lengths of *Linum crepitans,* a variety of the ordinary flax. They were measured, at a moment in which the growth had not yet ceased, * * *. The seeds were sown in a great deep flower-pot. Their number was purposely taken very high, so that they were *extremely crowded.* At starting, therefore, the difficulty for each seed was to get a root into the soil. It seems allowable to assume that all the seeds germinated. This has necessarily entailed an intense struggle, and many individuals must not have succeeded, or not sufficiently succeeded. For those who really got their root in the soil, there now came a good time. There was plenty of food for a good many of very small plants. The case, however, changed when the plants, becoming greater, required more room. Then a second struggle ensued, *viz.,* the struggle for the available food in the too narrow room. The plants now became more and more impeded in their growth'."

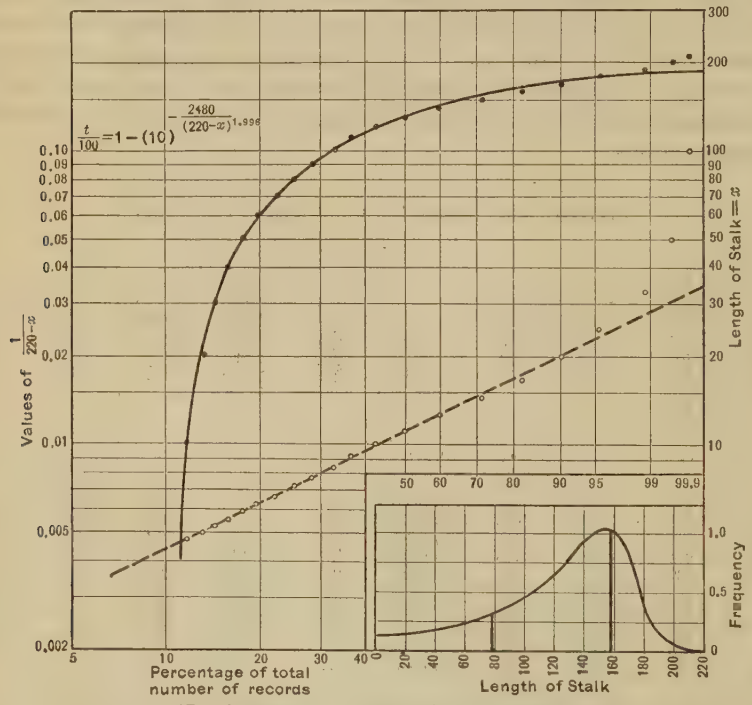

FIG. 17.—CURVES FOR LENGTHS OF STALKS OF FLAX

These facts are also shown by the curves and equations of Fig. 17 as follows: The difficulty in germinating is indicated by the fact that the frequency curve begins at a height of about 0.11 above the x-axis and by the form of the equation, which has a theoretical lower limit of $-\infty$, giving a finite frequency for a length of zero. The rapid early growth is shown by the fact that the frequency curve is nearly parallel to the axis at the left, which indicates strong reaction to the causes of growth. This was made

possible while the plants were small by the presence of plenty of food. Soon the confinement of the flower-pot and the depletion of the food in the soil began to retard growth; this is shown by the rise of the frequency curve. The large number of individuals between the lengths of 140 and 170 is shown by the height of the frequency curve at the mode and the steepness of the sides of the hump. Hence, for individuals of this size the reaction to the causes of growth was comparatively weak and the second struggle resulted.

While the departure of the duration curve from two or three of the points at the upper end of the curve would make it impossible to extend the curve in this direction, this departure applies only to about 1% of the total number of individuals observed and therefore does not detract from the value of the conclusions to be drawn from the curves as a whole. The close agreement of the theoretical curve with 99% of the data would lead to the conclusion that the equation is adequate for the purpose of an interpolation formula, and it is hoped that the method may prove useful in biology and statistics as well as in engineering investigations.

General Discussion

The foregoing illustrations show that a considerable variety of conditions governing the occurrence of phenomena which follow skew frequency laws of distribution can be investigated by the writer's simple methods. The cases illustrated cover such a general range of conditions that it would seem that any engineer could soon acquire facility in making frequency studies.

In previous papers and discussions published by the Society, numerous passages call attention to the limitations and difficulties in adapting Pearson's skew frequency curves for use with short series of observations of rainfall and run-off data, the problem which it has been the writer's object to overcome, if possible, by new methods of investigations. For example, Mr. Hazen states:*

"Data represented by skew probability curves are capable of being handled by mathematical analysis, but the methods are complicated. * * * for stream-flow data, there are two reasons why the mathematical analysis is not advisable: First, the difficulty of applying the methods is too great for some engineers; and, second, and more important, such short-term records as are available are not long enough to serve as a basis for a conclusive determination of the shape of the curve."

This undoubtedly refers to the proper type of the Pearson probability curve, the selection of which, therefore, often seems to be a matter of judgment and may be somewhat approximate, provided it fulfills the majority of the requirements. Five equations were required by Professor Pearson to cover the practical range of skew frequency variations.

Likewise, R. E. Horton, M. Am. Soc. C. E., states:†

"Tolley and others have shown that none of Professor Pearson's curves seems to be very satisfactory for representation of rainfall variation."

* * * * *

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 214.

† *Loc. cit.*, p. 228.

"The generalized law of error * * * like the Gaussian law of error, and most frequency curves utilized by biologists and statisticians, can only be integrated for particular cases. For this reason it is not wholly satisfactory for engineering work where integral frequencies, rather than probabilities of individual events, are mostly required * * *."

This criticism of the Pearson frequency equations, that they cannot generally be integrated for application to engineering problems, is entirely obviated by the use of the writer's equations.

Another serious objection to the use of the Pearson frequency curves is the uncertainty as to the accuracy of the constants when based on coefficients or criteria computed from a series of not more than twenty or thirty terms which show considerable irregularity and departure from a smooth curve. This is usually the case with observations of rainfall and run-off. In discussing the accuracy of the coefficient of skew which it is shown may be from 17.5 to 61.7% in error, for records from 20 years down to 10 years in length, Mr. Foster states:*

"An actual record will not correspond precisely to the theoretical curve; and the coefficients, as computed from an actual record, therefore, would probably show greater variations than are indicated in Table 3."

* * * * *

"(2) The coefficient of skew is greatly affected by the length of the series, and only a very long record would give a value for c_s, which would be considered very accurate."

In his closing discussion, Mr. Foster continues in the same vein:†

"For ordinary engineering work, however, the observations are very limited in number. As a result, although the coefficient of variation can be computed with a fair degree of precision, the coefficient of skew can be determined only approximately."

Messrs. Hazen and Hall,‡ and Mr. Frederic H. Hapgood,§ all bring out much the same idea in their discussions of Mr. Foster's paper.

In using the methods of plotting duration or integral frequency curves as described herein, and in obtaining the equations of these curves, it is not necessary to compute either of these coefficients unless they are desired for the purpose of classification or comparison.

That there are certain limitations and difficulties in applying the writer's equations, especially when both limits of the curve are finite, is admitted. However, these difficulties will be found to be more theoretical than practical, or else they will be found to be inherent in the very nature of such fluctuating phenomena as rainfall and run-off, and therefore would apply to all methods of analysis. Numerous trials have shown that curves plotted with these equations will fit data following skew frequency laws of distribution as closely as those based on any other equations heretofore published.

After developing the equations and method of plotting data on skew frequency paper, it was submitted to a member of the Society for suggestions and criticism. Through his kindness the writer received, more than a year

* *Transactions*, Am. Soc C. E., Vol. LXXXVII (1924), p. 166
† *Loc. cit.*, p. 197.
‡ *Loc. cit.*, p. 180.
§ *Loc. cit.*, p. 176.

after the formulas had been independently deduced, the little known but valuable and suggestive papers on skew frequency curves by Kapteyn and Van Uven.* They show that the function of R, which seems to be much the same herein as in their papers, indicates the character and effect of the causes producing the variations in the observed values of the phenomena. They state in the Second Paper (1) that when the records are distributed according to the "normal Gaussian law" (plot as a straight line on Hazen's probability paper), the "elementary increment† is independent of the value of R"; (2) when the data plot as a straight line on skew frequency paper (for example, Fig. 1), the lower limit of the curve is zero and $f(R) = R$ (Equation (S)), and "the elementary increment is proportional to the value of R"; (3) when the data, as plotted on skew frequency paper, fall on a curve (for example, Figs. 2 and 3) that can be made straight by the addition or subtraction of a constant (a in the foregoing discussion), "the elementary increment consists of a part proportional to the attained value of R (which is positive for a positive value of R), and of another part independent from R (which is negative for a positive value of a)"; and (4) if the data are best fitted by Equation (C), "the elementary increment consists of a part independent of R (which is positive for positive values of b) and of another part proportional to R (which is negative for positive values of R)".‡ It should be noted that the effects of the causes producing a finite lower limit are of opposite sign to the effects of causes producing a finite upper limit.

Equation (B) covers a condition common in hydraulic data and not discussed by Kapteyn and Van Uven. It provides for the simultaneous results from independent sets of causes of which some affect the lower limit and others, the upper limit. Equation (B) seems to be especially applicable to the mathematical expression of the variation in the frequencies of river discharges, when viewed in the light of the foregoing quotations. Consider the principal factors affecting variations in run-off. It is in general proportional to rainfall; a positive increment in the latter tends to produce a positive increment in the former. Absorption, deep seepage, and transpiration are all positive demands which are, more or less, both constant and independent of the amount of run-off and which have a most decided effect in reducing it (see Kapteyn and Van Uven's Deduction (3)). Evaporation from land and water surfaces depends in large part on the amount of moisture available, the

* "Skew Frequency Curves in Biology and Statistics" (First Paper), by Dr. J. C. Kapteyn, Professor of Astronomy at Groningen, 1903; "Skew Frequency Curves in Biology and Statistics" (Second Paper), by Dr. J. C. Kapteyn and Dr. M. J. Van Uven, Professor of Mathematics at Groningen, 1916.

† *Loc. cit.*, Second Paper, pp. 31–32, as follows:

"The rate of increase of some organism under the influence of some cause depends not only on the intensity of that cause, but also on the degree in which the organism reacts upon it. For example, in some cases the rate of increase of a plant may be considered to be proportional to the area of some organs of the plant. By measuring the diameter, x, of the organ concerned we find the rate of increase proportional not only to the intensity of the cause of growth (rain, solar heat, etc.), but also to the second power of the measured quantity, x. In this case we call x^2 the "reaction-function." Thus, the increment of the plant, particularly of the measured quantity, x, is proportional as well to the intensity of the cause as to the particular value of the reaction-function, $\Psi(x)$. * * *

"The increment within some period of growth may be considered as a multiple, m, of an elementary increment. This multiple being proportional to the intensity of the cause, the elementary increment is, in its turn, proportional to the reaction-function."

‡ In these quotations the nomenclature of Kapteyn and Van Uven has been changed to conform with that of this paper.

loss from this source increasing with an increase in run-off. At the same time topography has a large effect, while this cause of variation is entirely independent of any changes in the actual amount of the run-off (see Deduction (4)).

While this interpretation of the terms of the function of R may require modification, it is at least suggestive of a possible use that can be made in scientific investigations of such data. The plotting of two or three sets of annual flood heights or discharges to fit Equation (S) would seem to indicate that all but the causes producing effects proportional to R (see Deduction (2)) are of such minor importance as to be negligible. The writer's development of these equations was entirely empirical but the interpretation of $f(R)$, as expressing the results of the action of various causes affecting observed phenomena, gives them a semi-rational basis which materially increases their value.

In using these equations for the study of rainfall and run-off data, it is necessary, on account of the irregularity of the data, to assume the values of the upper and lower limits of the curve. This calls for the exercise of the engineer's judgment, guided by any indications as to a tendency toward definite limits when the data are plotted on "logarithmic" or skew frequency paper. This may be considered by some as an insurmountable objection to the use of these equations. It has been found, however, that two or three trials will generally give satisfactory results. It may be noted that Kapteyn and Van Uven also find it necessary to estimate the values of the limits for their curves.

If the data plot as a particularly smooth curve, especially at the lower end, a geometrical method* may be used by which the approximate values of the lower limit, a, of the curve can theoretically be determined. It is probable that a similar method could be used at the upper end of the curve to determine the value of the upper limit, b, if the data were sufficiently regular. All the values of the limits of the curves shown in the diagrams were found by trial with little trouble.

If sufficient data are available and the investigation is of considerable importance, approximate values for the constants of the equations can be determined by the graphical method here given and corrections to give more exact values can then be computed by the methods of least squares, but this should seldom be necessary.

Conclusions

1.—The graphical analysis of "integral skew frequency" or "duration curves", as herein presented, is developed especially for the examination of records consisting of from about ten to fifty observations. With fewer than ten the results are liable to great uncertainty, while with more than fifty the data must be grouped for convenience in plotting.

* See "Logarithmic Discharge Curves," in "King's Hand Book of Hydraulics," pp. 267–268.

2.—The equations presented cover practically the entire range of conditions necessary for the analysis of hydraulic and many other types of skew frequency data. The field is much more general than that heretofore available for practical use in engineering design.

3.—The equations may be useful in such studies as the growth of population.

4.—By the method suggested, data may be plotted as a linear function of the independent variable and hence the curve can be extended to a reasonable distance beyond the range of actual observations with much more confidence than if curves of either single or reverse curvature were required.

5.—Data may be plotted and curves drawn on "logarithmic" or "skew frequency" paper without the necessity of computing coefficients or criteria of uncertain accuracy, or the use of special tables of coefficients. The only mathematical aids required are an ordinary slide-rule and a table of four-place logarithms.

6.—The operations required by this method are so easily performed that for purposes of design any engineer can make scientific investigations of data which to the ordinary investigator seem to follow obscure skew frequency laws of distribution.

7.—With irregular hydraulic data, the accuracy of the method depends on the judgment and skill of the engineer in selecting the best values for the limits of the curves, and in drawing the line that best represents the plotted points. The other constants of the equations are found graphically or by simple computation.

8.—It is possible to analyze the causes of, and estimate their effects on, phenomena studied by the use of these equations, and to resolve complex results into simpler forms, which are more easily investigated.

Acknowledgments

This paper was prepared as the result of studies of the papers by Allen Hazen, M. Am. Soc. C. E., L. Standish Hall, Assoc. M. Am. Soc. C. E., and H. Alden Foster, Assoc. M. Am. Soc. C. E., and could not have been possible except for the previous studies of these engineers. Mr. Hall's courtesy and assistance has already been mentioned. Acknowledgment should also be made of the assistance of Horace W. King, M. Am. Soc. C. E., and Professor Theodore R. Running, both of the University of Michigan, who gave valuable suggestions and encouragement; also to E. P. Goodrich, M. Am. Soc. C. E., for information and data on population curves and for the papers by Kapteyn and Van Uven. Messrs. T. T. Lee, Chief Draftsman, and T. Kiang, former Assistant Engineer, Works Department, Chihli River Commission, assisted in the preparation of the diagrams.

DISCUSSION

H. Alden Foster,* Assoc. M. Am. Soc. C. E.—Perhaps the time is not far off when some enterprising engineer or philosopher will write a book on the theory of probabilities as applied to engineering and statistical work, in such a simple and unadorned manner that it will be easily understood by all engineers. Until then, the engineer who wishes to study data that are adapted to analysis by probability methods must learn how to do so through the papers published by the professional societies.

The author has made a new step in the discussion of engineering probabilities, and one which should be useful to the profession. The method of analyzing the data and preparing the diagrams differs from any heretofore suggested, and, undoubtedly, in certain respects, is a decided improvement over previous methods. All these methods consist in the selection of some form of smooth curve to be substituted for the irregular diagram given by the observed data. In some methods the curve is plotted entirely graphically. Other methods use a particular mathematical expression or equation, the constants of which are selected in such a way that the curve when plotted will best represent the observed quantities. In any case, the ultimate use of the curve may be divided into two classes: (1) to serve as an interpolation diagram by which the irregularities of a limited group of observations may be distributed in a logical manner; or (2) to extend the range of the observations considerably, either above or below the limits of the given data. This latter class is more common in engineering work; but the methods of analysis are equally well adapted to either purpose.

The author has developed a form of empirical equation which he uses to analyze the actual data. The constants in his equation are selected by an entirely graphical process. By an ingenious use of logarithmic plotting, he is enabled to bring the original data to the form of a straight line, provided, however, that the data actually agree with the general form of his empirical equation. In any actual case, an approximate selection of the constants is made, and a straight line is located, by eye, so as to pass through these points as closely as possible.

The form of equation which, heretofore, has been most used for this purpose is that developed by Professor Pearson† His equation is derived by mathematical reasoning from certain general assumptions as to the distribution of the data—these assumptions being based on the general shape of the frequency diagrams plotted from actual observations. The selection of the constants in Pearson's equations is made by a definite process, which is not influenced by the engineer's judgment except as will be mentioned. It is apparent, therefore, that Pearson's curves have a strong mathematical basis. Elderton‡ makes the following statement:

"Objection to Pearson's system has not been generally directed against its practical sufficiency, but has rather been founded on the contention that the

* With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.
† *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.
‡ "Frequency Curves and Correlation."

theoretical basis of the system is insufficient. It is interesting to note in this connection that in a paper read before the Statistical Society this year [presumably in 1906] Professor F. Y. Edgeworth, who has himself suggested other methods, points out that Pearson's Generalized Probability Curve appears more justifiable in the matter of *a priori* justification the longer its philosophic basis is subjected to criticism."

In spite of their sound mathematical basis, the Pearson equations are difficult to use in engineering work for two reasons: (1) the duration curve cannot in general be found because the frequency curve is in a form which cannot be integrated; and (2) the constants in the equation can be obtained, in general, only by taking the "fourth moments" of the observed data. The speaker* was able to overcome these objections for two types of the Pearson equations. These types are satisfactory for most classes of engineering data. The constants required for determining the equations can be readily obtained from the observed data by an entirely automatic process, the only difficulty being that, when the observations are quite limited in number, the computed values of the constants are subject to a certain amount of error.

The equations presented by the author do not have quite as sound a mathematical foundation as Pearson's equations. However, in their general shape, they satisfy the form of distribution of the data very well, and should be entirely suitable for practical use. The real test of the suitability of either system of curves is whether the data fit the plotted curve.

The selection of the method of analysis would probably depend on its relative simplicity and convenience. Mr. Goodrich's method could apparently be used to advantage for certain forms of distribution which do not fall in Type I or Type III of the Pearson curves, and which were not considered in the speaker's paper. In any case, the results obtained by one method should be about as accurate as those obtained by the other, and the precision of these results will be closely dependent on the number of items in the records used.

The use of ordinary probability paper, as originally proposed by Allen Hazen,† M. Am. Soc. C. E., has the advantage that it shows up clearly the relative skewness of any duration curve. Mr. Goodrich's method of plotting does not show this. For this reason, it might be advisable to replot the curves on the Hazen probability paper, even when they have been originally determined by the author's method.

The speaker is favorably impressed with the author's use of the frequency diagram, and especially with the analysis of this diagram as given by Dr. Kapteyn. The construction of the frequency curve, by the present method, should furnish material for a more complete analysis of the data. This subject is, of itself, rather involved, but should offer suggestions of value in certain cases. The method of analyzing population data is also instructive.

The coefficients of variation and of skew have certain advantages which, in the speaker's opinion, are not equalled by the new factors suggested by Mr. Goodrich. The coefficient of variation is a very definite measure of the "spread" of the observations, and can be readily visualized as the slope of the

* "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1666.

duration curve when plotted on the Hazen probability paper. The coefficient of skew, likewise, gives a direct measure for the skewness of the data, and can be visualized as indicating the percentage of time by which the mean differs from the median. The term, "average range", might be useful in some cases, but does not describe the distribution any better than the coefficient of variation. The term, "quartile ratio", seems to be introduced as a substitute for the coefficient of skew, but with no particular advantages over the latter. The chief advantage of the new terms is that they can be calculated readily after the duration curve is drawn.

Mr. Goodrich's method of analysis is quite ingenious, and he has done a service to the profession in making it available for general use.

A. Streiff,* M. Am. Soc. C. E. (by letter).—This paper amplifies and extends a series of papers on probability of rainfall and run-off and the application of skew frequency curves which reflects distinction on the Society. The writer believes that these thorough studies are unique and original, as similar investigations cannot be found in foreign engineering literature. This discussion will approach the subject-matter from a different viewpoint.

According to the ancient Greeks the supposed inability of man to peer into the future was caused by the curious Pandora; nevertheless, ever since then attempts have been made to solve that very problem, and in many instances with considerable success. Nor is any occultism involved; it is merely necessary to know the laws governing the event under consideration, and these, if found, will determine the future. In other words, it is merely necessary to describe accurately the subject under investigation as a function of dimensions, time, or other variables.

Sometimes such laws are known with accuracy and the forecasts, as for instance those of the astronomers, are of uncanny precision. Sometimes they are less definite, as in engineering, and still performance usually follows expectation. Again, a great many inseparable causes are at work, called chance, in which case probability methods are used and give utilizable results.

River flow and run-off have been regarded by several investigators as belonging in the latter class, and future expectations have been based on treatment according to the laws of chance. This, it is believed, should be modified. Of course, grouping ordinates according to size, and rectifying the integrated curve by varying the ordinate scale does not necessarily involve the theory of probability. That may be done with any function—the circle, the parabola, etc. A new geometry might thus be created and the properties in the "grouped ordinate" presentation might lead to some interesting correlations of the properties of figures.

If, however, from such graphical presentations conclusions are drawn as to future expectation there must be a certainty that the fundamentals of the probability theory are satisfied. For instance, a frequency curve for part of a circle might be drawn, but obviously no conclusion may be deduced from the mean value about the probable size of the next ordinate, for that is predetermined. Thus, when the author states that, "in other words, there is an

* Cons. Engr., Bangor Hydro-Elec. Co., Jackson, Mich.

even chance that the next succeeding record will fall within these limits", he should be certain that the river does not follow definite, separable laws, but is governed by chance. This, to be sure, has been the accepted view thus far. The assumption is indeed tempting. Nothing seems more variable than the factors which create river flow—precipitation, radiation, cloudiness, wind, barometric pressure, evaporation, transpiration, seepage, geologic formation, temperature, topography. How can the existence of separable laws be expected from such a multitude of variables? This, perhaps, explains why the subject has heretofore never been investigated.

On the other hand, indications that run-off does follow definite laws, may also be found. Professors A. E. Douglass,* of the University of Arizona, and Ellsworth Huntington,† of Yale University, have measured the variation in width of the growth rings of Sequoia trees and have found an oscillating variation around a mean value and a strong correlation with rainfall in some localities. Conclusions regarding the California climate for 3 000 years have been drawn. They also find correlation with solar activity, as expressed in the so-called Wolf numbers of sun spot frequency. In some instances, as in pine trees of the Eberswalde Forest, Germany, the correlation with Wolf numbers is exceedingly strong.

The remarkable fact involved is that such strong correlation with the Wolf numbers cannot be traced in the weather elements which influence tree growth, and the causes are as yet quite obscure. Professor Huntington believes that the tree acts as an integrator of the weather elements and adds the various influences not traceable separately. If, however, the limited surface of a tree acts as an integrator of the weather elements, how much more can this be expected from a river the drainage area of which covers many thousands of square miles?

Any one who has observed the path of rain storms realizes their fortuitous character. The drainage area, however, collects them all and will no doubt show continuous relations, if these exist. A casual inspection of run-off data dispels comparison with unrelated numbers. Hoyt‡ has shown that such data strongly converge around a constant mean. This also can easily be found in rainfall (see Table 17).

TABLE 17.—Percentages of Annual Rainfall at Lund, Sweden.

Average of:	Percentage of mean.	Average of:	Percentage of mean.
10 years	101.8	90 years	102.2
20 "	101.2	100 "	101.5
30 "	102.2	110 "	100.7
40 "	103.4	120 "	100.8
50 "	104.2	130 "	100.7
60 "	104.6	140 "	99.9
70 "	103.6	150 "	100.4
80 "	103.4	157 "	100

* "Climatic Cycles and Tree Growth," Carnegie Inst., Washington, D. C.

† "The Climatic Factor," Carnegie Inst., Washington, D. C.

‡ "Run-Off," by Gardner S. Williams, M. Am. Soc. C. E., in "Civil Engineer's Pocket Book."

Then the variation with the seasons also strongly suggests a controlled sequence. Likewise, every hydraulic engineer is familiar with the undulating appearance of a mass curve showing yearly run-off, indicating a periodic variation and grouping in series of wet and dry years. This latter quality is the cause of the skew frequency curve. The skew frequency curve is easy to imitate artificially, by assuming a definite function somewhat resembling a run-off curve. These run-off curves are periodic curves without definite period, that is, the values oscillate repeatedly around a mean value, but never repeat themselves. This is the class of curves analyzed by so-called periodogram analysis, which differs from harmonic analysis in that the latter is applied to curves which do have a definite period. A curve resembling the run-off curve can be obtained by taking a Fourier series. The word "resembling" should be emphasized, for the run-off curve cannot be analyzed by either harmonic or periodogram analysis, but requires special methods.

For the Fourier series:

$$y = c_0 + c_1 \sin x + c_2 \sin \left(2\,x - \frac{\pi}{2}\right)$$

in which, x and y are the co-ordinates, and c_0, c_1, and c_2, are constants. It may be found that this is a simple form of undulating periodic curve which has one greater and one smaller maximum per period, similar to a run-off curve with spring and fall maxima. This has a skew frequency curve precisely like those shown by the author. By adding a few terms or varying the constants the entire range of skew frequency curves can be obtained, which proves that these curves can be obtained under conditions of controlled sequence.

Furthermore, the probability theory furnishes several criteria proving that run-off figures are not unrelated numbers. The "Goutereau ratio" shows this. Goutereau* aided by Maillet has shown that the ratio: Mean variation divided by mean deviation equals 1.41 for Gaussian numbers. Flow data taken at random do not substantiate this ratio. The 53-year record of the Murray River, Australia, given by L. Standish Hall, Assoc. M. Am. Soc. C. E.,† furnishes a ratio of only 1.016. Marvin has recently emphasized the value of this index.‡

There is another criterion. If an event has occurred a times and missed b times in n cases, the probability *a posteriori*, $\frac{a}{n}$, is also the adopted value for the probability *a priori*. A deviation, e_1, from $\frac{a}{n}$ is to be expected with a probability, $f(g)$, according to the relations,

$$g = e_1 \sqrt{\frac{n^3}{2\,a\,b}}$$

and

$$f(g) = \frac{2}{\sqrt{\pi}} \int_0^g e^{-t^2}\,d\,t$$

* "Sur la variabilité de la température," *Annuaire* de la Société Météorologique de France, 1906.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 246, Table 16.

‡ *Monthly Weather Review*, September, 1924.

This is the theorem of Bayes. Taking, for instance, $n = 365$ and $a = 104$, $e_1 = \frac{1}{20}$. Then, $g = 1.5$ and $f(g) = 0.966$; or the chances are 28.4 to 1 that $\frac{a}{n}$ equals $\frac{104}{365}$ for the next 365 unrelated numbers, within 5 per cent. This obviously is not true for rivers, for it all depends on a wet or dry year whether, for instance, the daily gauge reading will be lower than a certain height. Actual count on the Tennessee for gauge heights below the reading, 1, gave the data of Table 18. Thus, in 29 trials the count gives 2 "yes" against 27 "no", whereas the theorem of Bayes gives 28 "yes" against 1 "no". If the theorem of Bayes is true, the river flow cannot be a fortuitous sequence.

For twenty-five years the writer has cherished the idea of deducing definite laws of river flow, and for the past ten years has devoted a great deal of work to this problem. The question appeared not to be readily solvable, for rivers would yield no results if treated with the existing means of periodogram analysis. It became necessary to devise special methods in order to obtain results, until finally the laws of river flow were found.

The subject is too voluminous to be presented fully in this discussion. Briefly, the main laws of river flow are as follows:*

1.—Hydrographs can be resolved into a definite number of elementary curves, which are either homogeneous or damped oscillations.

2.—The same elements recur in different rivers, but the phase and amplitude change.

3.—These elements may be divided in two groups. There are no others.

4.—The first and most important as to size is due to the earth's motions. This group does not change the mean yearly run-off from year to year.

5.—The second group causes the variation from year to year. This group is identical with elements in the curve of the Wolf numbers of sun-spot frequency.

6.—Run-off follows the long climatic cycle described by Brückner,† the so-called Brückner cycle, which is identical with the second largest element in the curve of the Wolf numbers. The low levels in the Great Lakes at present are produced by the same cycle.

TABLE 18.—Frequency of Daily Gauge Readings Below 1 Foot, Tennessee River at Florence, Ala.

Year.	Frequency.	Year.	Frequency.	Year.	Frequency.
1881	104	1890	0	1900	97
1882	22	1891	0	1901	80
1883	37	1892	0	1902	101
1884	33	1893	9	1903	78
1885	43	1894	4	1904	147
1886	6	1895	84	1905	88
1887	69	1896	123	1906	42
1888	95	1897	73	1907	5
1889	0	1898	90	1908	41
		1899	63		

* "On the Investigation of Cycles and the Relation of the Brückner and Solar Cycle," by A. Streiff, *Monthly Weather Review*, July, 1926.

† 'Klimaschwankungen seit 1700," Vienna, Austria, 1890.

In view of these laws the sequence of run-off years cannot be regarded as fortuitous. For example, in Southern Michigan increased run-off is expected for the next two years, in accordance with the strong correlation with the Wolf numbers. This conclusion of course involves the use of probability, which is, therefore, not to be dispensed with, but merely modified in its application by laws of river flow. Nor is the clever presentation of the author thereby impaired. It is possible that the relation between extreme values remains the same in the original data and its "grouped" presentation, regardless of the fact that the data are values of a definite function. This point will no doubt be clarified in due time.

It is impossible to draw conclusions as to the expectancy of next year's run-off; but this point is of little consequence as far as the application of the frequency curve is concerned, for its chief use lies in the estimate of maximum and minimum conditions. These are obtained directly and neatly by resolution into cycles. That phase combination which produces a maximum or minimum can easily be determined by mere inspection, and this gives the desired quantity by means of a short record of twelve years. A correction for the longer cycles, namely, the Brückner and the longest so-called secular cycle (which are, compared with the shorter cycles, very flat), can be estimated. Their influence is not material from year to year, and is of real importance only in cumulative effects, such as lake levels.

It would appear at first improbable that extreme values can be determined from a record of only twelve years, but it is easily explained as follows: The elements of river flow are incommensurable, as, for instance, $\sin x$ and $\sin x\sqrt{2}$ and, therefore, a long time may elapse until the elements are combined in nearly the hand-picked combination which produces the maximum or minimum. This constitutes the great value of the writer's method over the frequency curve, for it is hardly possible with the latter to obtain a clue regarding maxima or minima with a twelve-year record. A resolution into cycles will accomplish just that.

The foregoing does not apply to flood flows—this has not been investigated, at least not by the writer. For the present it applies only to monthly and yearly averages. Eventually the flood flows too will perhaps yield to such treatment and expose the laws under which they occur.

W. A. Shewhart,* Esq. (by letter).—Some very broad claims are made for the method of analyzing data proposed by Mr. Goodrich. Hence it may be of interest to consider some illustrations in which these claims do not appear to be justified and then to consider a few general criticisms of the method. Both the case where the number of observations is small and where it is large will be considered.

Table 19 gives the results of six drawings of samples of 10 made under known conditions. In this case the quantity, X, corresponds to run-off in Mr. Goodrich's paper. The endeavor will be to find the best straight line to

* Bell Telephone Laboratories, Inc., New York, N. Y.

represent the distributions from which the samples were drawn. Figs. 18 and 19 show the corresponding ogives* plotted on the special co-ordinate paper suggested by Mr. Goodrich.

TABLE 19.—VALUES OF X.

Sample 1.	Sample 2.	Sample 3.	Sample 4.	Sample 5.	Sample 6.
15	18	18	42	22	22
26	28	17	43	31	29
33	35	27	43	33	33
42	42	34	44	34	36
45	48	35	44	41	37
55	48	35	44	44	41
57	49	39	44	45	43
61	53	42	45	48	47
62	58	57	54	50	47
67	70	62	57	57	55

Samples 1 and 2 appear to fall approximately on the same straight line which is almost parallel to that representing Sample 3. This would indicate similar types of parent distributions with the median value for Samples 1 and 2 greater than that for Sample 3. In a similar manner Samples 5 and 6 appear to have come from the same parent distribution or universe, whereas the data for Sample 4 lie on a curve which is concave upward, indicating that they must have come from a universe different from that from which Samples 5 and 6 were drawn.

Now, the truth is that Samples 1, 2, and 3 were drawn from the same rectangular distribution,

$$y_1 = 2 \text{ for } 5 \leq X \leq 75;\ y = 0 \text{ elsewhere}$$

and Samples 4, 5, and 6 were drawn from a normal universe,

$$y_2 = \frac{1}{\sigma \sqrt{2\pi}}\, e^{-\frac{(X-m)^2}{2\sigma^2}}$$

in which, $m = 40$ and $\sigma = 10$. Hence, neither of the parent distributions should plot as a straight line on the paper suggested by Mr. Goodrich. The heavy curves in Figs. 18 and 19 show the integral curves for the two parent distributions. Obviously, the proposed graphical method does not give very reliable information in this case.

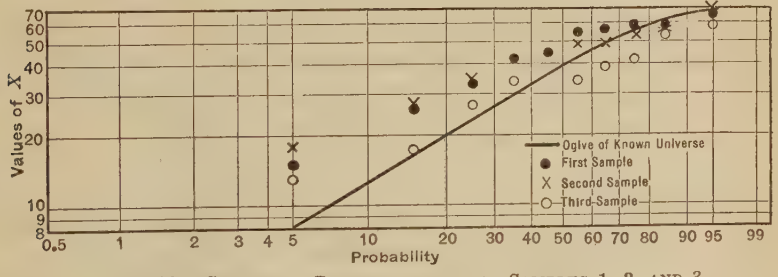

FIG. 18.—GRAPHICAL REPRESENTATION OF SAMPLES 1, 2, AND 3.

What will happen when the data are plotted on co-ordinate papers for which the observed curves should be straight lines? Of course, the ogives for the first three samples should plot as straight lines on ordinary co-ordinate

* An ogive curve is an integral curve.

paper, Fig. 20, and the other three should plot as straight lines on normal law probability paper, Fig. 21. Here again, however, is found what may be expected—samples of Size 10 are not sufficient to determine straight lines even when the right kind of co-ordinate paper is used.

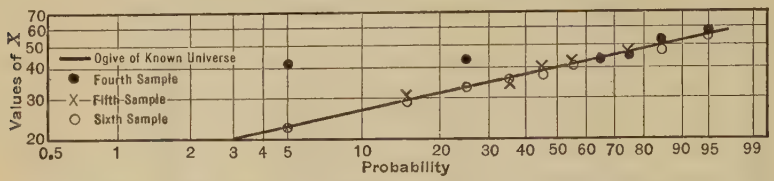

FIG. 19.—GRAPHICAL REPRESENTATION OF SAMPLES 4, 5, AND 6.

Similar instances of the failure of the method could be shown for samples as large as, or larger than, 50, hence it does not appear that the conclusion is justified, namely, that "the graphical analysis of 'integral skew frequency' or 'duration curves', as herein presented, is developed especially for the examination of records consisting of from 10 to 50 observations".

The author states that his method was developed to overcome some of the limitations of Pearson's method of fitting skew frequency distributions. For example, he makes specific mention of the fact that it is well recognized that Pearson's method breaks down for small numbers of observations, because the errors in the third and higher moments are so large. Now, the method under review is supposed to be especially adapted to the analysis of such short series of data, because a straight line can easily be found to represent the data. Already certain cases have been demonstrated in which the method appears to break down. Furthermore, a little consideration shows that such a breakdown is to be expected because large errors of sampling exist in the estimates of the parameters required to fix the straight line on the special co-ordinate paper proposed by the author when n is small, just as large errors exist in the estimates of the parameters used to determine the shape of a frequency curve under similar conditions.

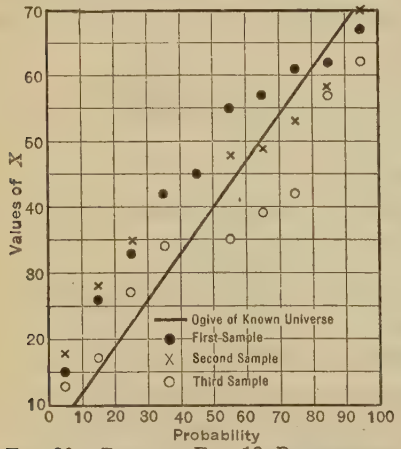

FIG. 20.—DATA OF FIG. 18 PLOTTED ON PAPER FOR WHICH THE OBSERVED POINTS SHOULD FALL ON THE HEAVY LINE.

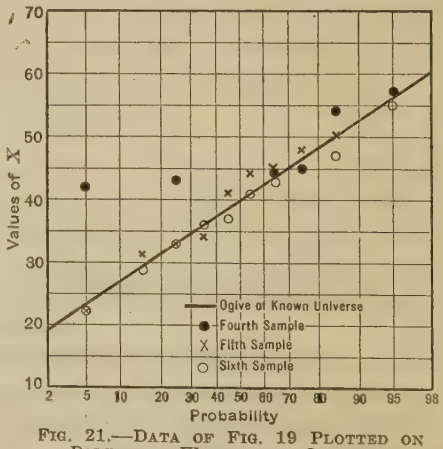

FIG. 21.—DATA OF FIG. 19 PLOTTED ON PAPER FOR WHICH THE OBSERVED POINTS SHOULD FALL ON THE HEAVY LINE.

One may go even further, however, than citing particular cases where the method proposed by Mr. Goodrich fails. Let us suppose that the type of the parent distribution is known and all that the graphical method is required to do is to give an estimate of the parameters. Let us find the best straight line representing a series of n observed data drawn from a normal universe,

$$y_3 = \frac{1}{\sigma' \sqrt{2\pi}} e^{-\frac{(X-m')^2}{2\sigma'^2}}$$

in which, m' and σ' are unknown. In order to determine the probability that a value of X will lie within the range, X to $X + dX$, estimates must be found of both the parameters, m' and σ'. Of course, the median of the ogive is an estimate of m' and the semi-quartile range (used by Mr. Goodrich) divided by 2 (0.6745) is an estimate of σ'.*

Now, it is well known† that the error of the median is 1.253 times as large as that of the arithmetic mean, $\sum_{i=1}^{n} \frac{X_i}{n}$, and that the error of the previous estimate σ_1 of σ is 1.65 times as large as that of the customary estimate, $\sum_{i=1}^{n} \sqrt{\frac{(X_1 - X_{ar})^2}{n}}$, in which, X_{ar} is the average value of X. Hence, the location of the center of the distribution (by the ogive method) in Equation (3) by $1.57n$-observations is no better than that obtained by the arithmetic mean of n-observations or, in other words, the ogive method is only 63.7% efficient for the determination of the mean. In a similar manner the estimate of the standard deviation by the proposed method is only 36.8% efficient.‡

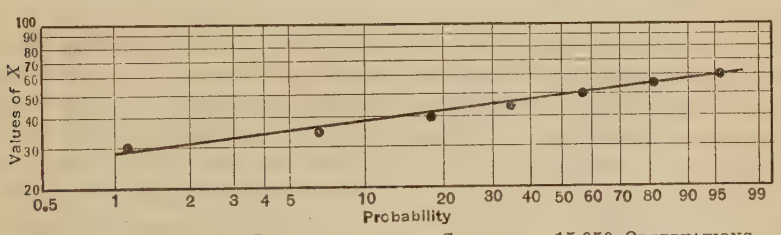

FIG. 22.—OBSERVED DISTRIBUTION OF A SAMPLE OF 15 050 OBSERVATIONS.

There is also another inherent error in the graphical method for the case in hand. Since the slope of the line on normal probability paper is given by the observed standard deviation, and since the most probable observed standard deviation must be multiplied by $\sqrt{\frac{n}{n-2}}$ to get the most probable standard deviation of the universe, it is obvious that the most probable observed slope determined by a small sample will be less than the true standard deviation of

* Of course, the slope of the straight line drawn through the points representing the observed data is also an estimate of σ'

† "On the Probable Errors of Frequency Constants," by Karl Pearson, Biometrika, Vol. XIII, Pt. 1, pp. 113–132.

‡ The efficiency of 64.9% could be obtained by estimating σ' from the ogive range of $\frac{100}{14}$ to $\frac{1\,300}{14}$ per cent.

the universe.* It will be noted that the slopes of the observed ogives are, in general, less than those for the true inverses, as is to be expected.

It has been seen that certain criticisms of the proposed method suggest themselves when the sample is small. Next, a typical case will be considered, in which the results of the analysis of a large sample by the graphical method fails to give the desired information whereas standard methods of analysis prove satisfactory. The distribution of a sample of 15 050 observations is plotted in Fig. 22. The straight line appears to fit the data quite as well as some of the illustrations given by Mr. Goodrich and yet customary analytical methods show almost beyond a doubt that the observed data do not constitute a random sample. The analysis of these data by customary methods has been published.†.

It is hoped that this discussion will help to point out some of the pitfalls in attempting to draw conclusions from small samples. In the light of this discussion it also seems that several of the broad claims for the proposed graphical method should be somewhat modified. It also appears that much more work must be done before the new method can be expected to replace the more reliable standard analytical methods either for small or large samples.

W. H. R. Nimmo,‡ Assoc. M. Am. Soc. C. E. (by letter).—This paper is a valuable addition to the previous papers on the same subject by other members of the Society. The comparatively simple method of treating frequency data developed by the author should prove of great value to hydraulic engineers in the solution of their problems. The writer has been able to make immediate practical application of the author's methods to his own work.

Early in 1918 the Government of Tasmania decided to develop the full capacity of the Great Lake for the generation of power. This lake has a catchment area of 153 sq. miles, including the water surface of 42 sq. miles at the natural winter level. By constructing a dam about 1 200 ft. long at the outlet at the south end of the lake, it was possible to obtain ample storage capacity to completely regulate the run-off from the lake and also from streams which could be diverted into it. The lake is at an elevation of 3 300 ft., and parts of the catchment basin rise to more than 4 000 ft. above sea level.

A more or less complete record of rainfall had been kept for 28 years at the dam site. Owing to frequent snowfalls the country is not continuously inhabited and the rainfall observations at other places in the drainage area were limited to partly complete records of less than 2 years' duration. It was known, however, that the precipitation at the north end of the area was from two to three times that at the dam site. Records of evaporation from a 3-ft. land pan, situated near sea level at Hobart, distant 80 miles from the lake, were available for 8 years. Daily records of the level of the lake and the discharge therefrom were available for one complete

* "Correction of Data for Errors of Averages Obtained from Small Samples," by W. A. Shewhart, *Bell Technical Journal*, April, 1926.

† "Quality Control Charts," by W. A. Shewhart, *Bell Technical Journal*, October, 1926.

‡ Chf. Draftsman, Main Roads Comm., Brisbane, Queensland, Australia.

year only, and no continuous gaugings had been made of any other stream in the State.

It fell to the lot of the writer to determine, from this meager data, supplemented by gaugings of rivers on the mainland of Australia, the probable total mean annual run-off from the lake and the storage capacity required to regulate it completely. The investigation resulted in the development of a formula for the annual run-off indicating that the maximum continuous yield would be 235 000 acre-ft. per year, equivalent to 324 sec-ft. To provide for the storage of water diverted from adjacent areas, a dam has been constructed to impound 35 ft. of water, creating a storage of 1 150 000 acre-ft. having a water surface of 60 sq. miles at full supply level.

TABLE 20.—Run-Off of the Great Lake, Tasmania.

Year. (1)	Run-off, in thousands of acre-feet per year, R. (2)	Percentage of time, t. (3)	$R - 120$. (4)
1925	134	5.5	14
1918	153	16.7	33
1919	156	27.8	36
1920	166	38.9	46
1922	205	50.0	85
1921	218	61.2	98
1924	308	72.2	188
1923	348	83.3	228
1917	356	94.5	236

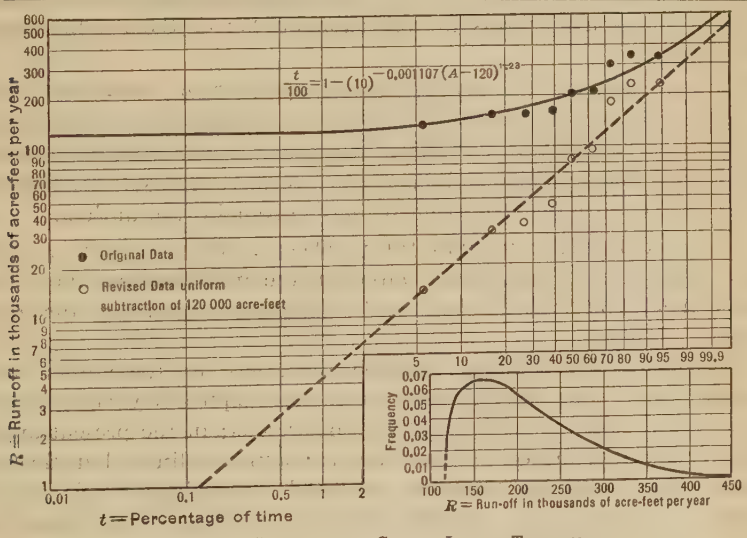
$\frac{t}{100}=1-(10)^{-0.001107(A-120)^{1.23}}$
Original Data
Revised Data uniform subtraction of 120 000 acre-feet
R = Run-off in thousands of acre-feet per year
t = Percentage of time
Frequency
R = Run-off in thousands of acre-feet per year

Fig. 23.—Curves for Great Lake, Tasmania.

As the records of run-off for each year since 1918 have become available, it has been a source of much interest to the writer to compare his estimated discharges with the actual outflow from the lake. Records for nine years are now available, including 1925 which was a year of very low run-off, and for this period the estimated yield exceeds the measured run-off by

16 per cent. The data for applying the author's method to the 9-year record of run-off are given in Table 20 and the corresponding curves are shown on Fig. 23. From the curves the following values are obtained, namely, $a = 120$, $c = 1.23$, and $h = 0.001107$. Substituting these values in the author's Equation (5), the probable mean annual run-off is found to be 238 000 acre-ft. per year, equivalent to 328 sec-ft., which agrees closely with the writer's estimate of 324 sec-ft.

JOHN TUCKER, JR.,* ESQ. (by letter).—The general subject of the study of statistical frequency data is an interesting one and apparently is becoming of more and more use to the engineer. Any new and original methods of the analysis of the data as those proposed by the author are of interest and may prove of considerable value. There are, however, some criticisms to be made of the paper.

The author objects to the use of the Pearson frequency curves because of the uncertainty of the accuracy of the constants when based on coefficients computed from a series of not more than twenty or thirty terms. It should be carefully noted that the curves obtained by the author will not approximate the curve attained from an infinite universe (that is, a very large amount of data) any closer than the Pearson curves obtained from the same data. The precision of the Pearson parameters, β_1 and β_2, may be expressed by their standard deviation.† The probability of the data indicating any of the Pearson types of curves is, therefore, definitely determined. Mr. Goodrich does not express the precision of any of the results obtained by his methods.

The author calls for the assumption of an upper or lower limit to the curve. This method, of course, invalidates the usefulness of the analysis, and calls for an extrapolation process. It is almost unnecesary to state that this is the most undependable and dangerous process to apply in mathematics.

Referring to the numbered "Conclusions" given by the author:

1.—It is not necessary to limit the number of observations in graphical plotting. As stated by the author, although the individual points will not appear distinctly, the curve is indicated with greater accuracy, the greater the number of points.

3.—The use of the graphical methods developed by the author are of very small value in statistical studies or in any application where extrapolation is applied. Extrapolation is always dangerous; in the present application the danger is removed by assuming a maximum limit, but the value and force of the method are thereby destroyed. Examination of several of the diagrams will show the indefiniteness of the curve form. In Fig. 8 neither branch is definitely indicated, the left branch being more definite than the right. In Fig. 9 the left branch should be a constant frequency, not the continuation of the given straight line. The right-hand branch of Fig. 17 is another forceful illustration.

* Washington, D. C.

† Statistical Methods for Research Workers" (Biological Monograph), by R. A. Fisher.

4.—The reason for assuming that extrapolation by means of the author's graphical straight-line methods is more reliable than extrapolation of a curve of double curvature as implied by him, is not very apparent, especially since the plotted points at the ends of several of the curves indicate a trend away from the plotted straight line.

5.—Frequency curve paper may be used with equal effect and with equally satisfactory results as the logarithmic or skew frequency paper, in the writer's opinion.

6.—It is true that any engineer may follow and apply the methods of the paper, but it is absurd to claim that he can make scientific investigation of frequency data by any method unless he has had a rather rigorous training in statistical mathematics. In no other branch of elementary mathematics is error in methods and in application so imminent.

7.—Statistical methods are developed for the best possible determination of unknowns. If the results depend on the skill and judgment of the engineer then the method loses its force and value; thus skill and judgment become the sole necessity, and the statistical methods should be discarded.

L. Standish Hall,* Assoc. M. Am. Soc. C. E. (by letter).—The author has made a valuable contribution to the subject of skew frequency curves as applied to engineering data. He deserves especial credit for having developed an entirely original method of treating such curves. A study of the examples given leaves no room for doubt that the method presented will yield satisfactory results as far as determining curves that will fit a given set of data is concerned.

It is the writer's opinion that many of the variations in the types of frequency curves encountered in run-off data are possibly of an accidental nature and are due to the shortness of the records available, or what statisticians term "fluctuations of sampling". It would appear that if some system of frequency curves is adopted, having a sound mathematical foundation, such curves could be used as a means of comparing and correlating the various data. In this way the data from many streams could be compared and more satisfactory results would be possible than could be reached from the uncorrelated study of individual streams.

For this reason the writer is inclined to favor the Pearsonian frequency curves despite the difficulties of their application to hydraulic data. These difficulties are not due to any fault of Professor Pearson's theory, but are largely caused by the shortness of the records which must be used in applying the curves. The variation in the $c\ s$ (attention to which has been called by the author) is merely an indication that the data used are of insufficient length to determine accurately the frequencies to be expected. The omission of this coefficient from the computations does not alter this condition, which is inherent in the data.

One important feature favoring the use of the $c\ v$ and the $c\ s$ is the fact that for a given type of curve, data having the same $c\ s$ will have variations

* Chf. Hydrographer, East Bay Municipal Utility Dist., Oakland, Calif.

from the mean proportional to the $c\,v$.* This fact is especially important in the correlation of data from different streams.

Furthermore, a study of the $c\,s$ and $c\,v$ of streams having long records, indicates that a more or less definite relation exists between these two coefficients.† It is possible that with the extension of stream-flow records, the correlation between these coefficients can be brought within narrower limits.

It appears essential to the writer that some measure be had of the reliability of the frequency curves drawn either by the author's method, or by any other method. Such a measure can be had.‡ If n is the number of terms in a series, p, the probability of an event occurring, and q, the probability of the event failing, the probable error of the event occurring is $e = 0.675 \sqrt{n\,p\,q}$. The probable error reduced to percentage of time would be, $e_t = \frac{e}{n} \times 100$. In Table 21 are given values of e_t for varying values of p, q, and n.

TABLE 21.

p.	q.	$0.675 \sqrt{p\,q}$	Values of e_t.			
			$n = 10$.	$n = 25$.	$n = 50$.	$n = 100$.
0.01	0.99	0.067	2.1	1.3	0.9	0.6
0.05	0.95	0.147	4.6	2.9	2.1	1.4
0.10	0.90	0.203	6.4	4.1	2.9	2.0
0.20	0.80	0.270	8.5	5.4	3.8	2.7
0.30	0.70	0.309	9.8	6.2	4.4	3.1
0.40	0.60	0.331	10.5	6.6	4.7	3.3
0.50	0.50	0.337	10.7	6.7	4.8	3.4

Elderton states:

"The general rule followed by statisticians when considering probable errors is that unless a result exceeds the expected by two or three times the probable error, it is not safe to assume that the particular case differs from the expected result."

To apply the results in Table 21 to a specific case, assume that the frequency curve drawn from a 10-year record shows that a run-off of 40% of the mean, or less, is to be expected 10% of the time. The probable error shows that in 50% of the cases, the run-off as determined would occur between 3.6 and 16.4% of the time. Considering the extreme case of three times the probable error, the run-off might never occur, or it might occur as often as once in 3½ years. If the curve had been determined from a 100-year record the probable error would be reduced from 6.4 to 2.0 per cent. Applying this to the case just considered, the run-off of 40% of the mean, or less, would probably occur between 8 and 12% of the time, or in the extreme case between 4 and 16% of the time. Thus, Table 21 should permit the engineer to determine the limits within which his frequency curve may be expected to vary.

The writer is greatly interested in the curves for the Wimmera River (Fig. 11) as indicating the flexibility of the author's general equation. It

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 162 and 196.

† *Loc. cit.*, Vol. LXXXIV (1921), pp. 248-249.

‡ "Frequency Curves and Correlation," by W. Palin Elderton, p. 132 *et seq.*

would be of interest if the author would explain how the values of the five constants in this equation were determined.

Perhaps the most important application of probability methods is the one first made by Allen Hazen,* M. Am. Soc. C. E. It is believed that, by the use of recent developments in the methods of treating skew frequency curves, improvements in the application of probability methods to storage could be made and much more reliable results secured.

E. P. Goodrich,† M. Am. Soc. C. E.—All engineers are familiar with the fact that observations with reference to any quantity, whether it be the measurement of a single distance, or data with reference to rainfall or run-off, or the petals of a flower, or of other biological, sociological, and engineering factors, when plotted side by side, will form a series of points which have an ogee shape. Such a curve may be called a distribution curve.

If the items are gathered together, so that the number of each size between progressive units of measurement are grouped, the result is a curve which may be called a frequency curve. A number of general formulas have been found to fit those two curves more or less accurately, depending on the character of the observations, the conditions which underlie the variation in the quantity itself, and other features.

The frequency of occurrence of engineering, sociological, and biological phenomena may, but generally does not, correspond with the frequency at which deviations have been found to occur in the repeated measurement of a fixed physical quantity. The curve which represents the frequency of occurrence of given deviations from the arithmetical mean of such measurements, shown either in quantity units or in percentages of the average, has been found by experience to agree closely with the mathematical curve of which the demonstration was first published by Adrian in 1808 at Philadelphia, Pa., in *The Analyst*,‡ generally known as Gauss' "Error Law". Gauss' derivation was published in 1809,§ but he is said to have stated that he had used it since 1794. In 1837 Hagen gave another method of analysis which has been generalized by Eggington.‖

Fisher¶ states that "* * * among the leading mathematicians of the present time there exists no uniform opinion as to the truth of the axioms underlying the theory of probabilities."

Brunt states that:**

"The proofs of the Normal Error Law * * * are based on certain definite hypotheses concerning the nature of accidental errors. It has been shown that,

* "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply." *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† Cons. Engr., New York, N. Y. This discussion by Mr. E. P. Goodrich is largely subject matter of a talk given by him in presenting this paper for R. D. Goodrich, M. Am. Soc. C. E., at the meeting of the Society on September 8, 1926. Much of the material was provided by Mr. R. D. Goodrich covering results of his studies subsequent to the submission of the paper.

‡ See p. 209, "Practical Least Squares," Leland, McGraw-Hill, 1921.

§ *Loc. cit.*, p. 201.

‖ See p. 15, "The Combination of Observations," Brunt, Cambridge Press, 1917.

¶ See p. 127, "The Mathematical Theory of Probabilities," Fisher.

** See p. 17, "The Combination of Observations," Brunt, Cambridge Press, 1917.

if accidental errors to which a series of observations is liable satisfy these hypotheses, the errors of observation will be distributed according to the normal law. The final justification of the use of Gauss's Error Curve rests upon the fact that it works well in practice, and yields curves which in very many cases agree very closely with the observed frequency curves. The normal law is to be regarded as *proved by experiment,* and *explained* by Hagen's hypothesis. When the curve of frequency of the actual errors is not of the form of the normal curve, we may safely conclude that the nature of the accidental errors concerned is not in accordance with Hagen's hypothesis."

The normal curve has applications in a region where deviations from a mean value are considered, although these deviations are not, properly speaking, of the same nature as accidental errors of observation; for example, it is frequently applied in biological questions. The use of the normal curve in such cases is justified only when the differences between individual cases are produced by causes the mode of action of which is in accordance with Hagen's hypothesis.

In other words, any one can propose a series of formulas with their corresponding curves; and if they fit the facts better than any other formulas, the one which fits best and is easiest of application is probably the most desirable one to use. That is the only reason why the author suggests his formula, because after testing it in a great many cases, he finds with reference to hydraulic data, for example, that it works better than that of Pearson, which has been the formula most used by engineers thus far. In biological data it fits just as well as Kapteyn's or Pearson's formula in all the cases the author has been able to discover.

The test of "goodness of fit" is made by plotting the curves by the two methods, computing the differences, finding their variation, and measuring it by the usual statistical method of computing the standard variation, or other characteristic measure.

There is a further reason why the author suggests the use of his curve—because it is almost exclusively graphical in its detail of application. The simple plotting of the data on the type of skew frequency paper that he has devised makes it possible for any engineer to apply the methods described, at least after a little practice. Furthermore, as stated by a number of those who recently have discussed the other papers which have been written on the subject of frequency and probabilities, it is difficult except in special cases to integrate the frequency curve when the equation is given in the Pearson form. In the author's formula the mathematical relationship between the two curves is obvious and the investigator can pass directly from one to the other without going through the detail of an integration or differentiation. In other words, when one is accustomed to the use of the two formulas he can write them out immediately as soon as the data are secured.

The constants which are found in the general form of the curve appear to have specific meanings, which is not the case with reference to any other curves except possibly those of Kapteyn. That is, a is the limit in one direction; b is the limit in the other direction; c, or c divided by d, meas-

ures the skewness of the curve; and the factor, h, depends on the system of logarithms used, the exponent varying with the unit of measurement of the data, whether it be "feet" or "meters", "inches per acre", or other unit.

Another reason why the author's formula and method are more adaptable than the others is that it is usable with relatively fewer observations. If one undertakes to compute the different factors involved in Pearson's equation, for example, he discovers that a large number of observations is necessary before anything like real accuracy can be obtained. In fact, the distribution curve, the frequency curve, the first moment curve, and the second moment curve must either be computed or actually graphed and measured, in order to discover the four constants which Pearson usually employs.

In the author's formula the whole method operates in the other direction. The investigator plots his data and from these computes the factors or constants directly. Having determined the relative percentage of time or number of observations that correspond with each proportional group he can work directly on the special paper devised by the author. The first plotting will take some one of a series of typical shapes. Of course, it will be exceptional that the points will follow precisely along any one of the type lines, but the several groups which are indicated in Fig. 24 are typical of the types of data which may be expected.

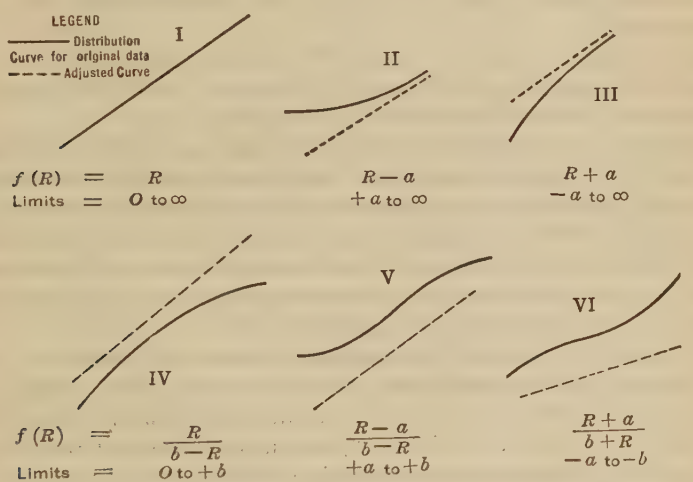
LEGEND
Distribution
Curve for original data
Adjusted Curve
I
II
III
f (R) = R R − a R + a
Limits = O to ∞ + a to ∞ − a to ∞
IV
V
VI
f (R) = R/(b − R) (R − a)/(b − R) (R + a)/(b + R)
Limits = O to + b + a to + b − a to − b

FIG. 24.—TYPES OF DISTRIBUTION CURVES AND CORRESPONDING FUNCTIONS OF THE VARIABLE FOR GOODRICH FORMULA.

The author discovered that, for each type of curve, after the data are adjusted by use of proper constants with appropriate limits as to the observations, the adapted function will finally plot as a straight line. In Type I the original data plot as a straight line and the frequency curve is very close to the normal curve of error, as is seen in Fig. 1. Again, in Type II, exemplified in Fig. 2, it is to be noted that as the curve extends toward the left it becomes flatter and flatter; and if it were indefinitely extended it is easily conceivable that it would become horizontal at about 8 on the vertical

scale; in other words, 8 is the limit to be set in the value of a. It is also to be noted in the equation given in Fig. 2 that $(R - 8)$ is the actual quantity used. In other words, the plotting of the curve often determines the actual limits of the data. This simply means that to a certain point—a distance, a, on the measurement—there is no precipitation each year, or a certain minimum number of petals of a flower, or a certain number of deaths per annum; and after the measurement, a, is passed then the variation is approximately in accordance with the normal law of frequency. Type III differs from Type II in having the theoretical lower limit on the negative side of the origin so that a finite frequency occurs when the value of the variable is reduced to zero.

In the next curve, Type (IV), instead of being a subtractive quantity, the limit is above the data, and the difference between the observed figures and the limit is the quantity that corresponds with this law of frequency. The limits for this form are much more difficult to determine from observation of the plotted data. Few curves of this type are discovered.

When it comes to Type V it is seen from Fig. 8 that as the curve extends toward the left it approaches 70 as its limit, and as it extends toward the right it is approaching 150 as its limit. In other words, if one started by drawing a curve which would fairly correspond with the observations, the limits within which this variation occurs would be indicated; and the formula evolved in this particular instance gives $(R - 70)$ as the numerator and $(150 - R)$ as the denominator.

When it comes to the final type (VI), which is very rare, it is usually necessary to select points which appear to represent the curvature fairly well, and solve some relatively simple equations to obtain the limits.

After having tentatively determined the apparent limits, it is sometimes found necessary to make a second or third approximation. Having secured an adjusted line from the proper selection of the limits, the value of c, which is the exponent of the function, is the tangent of the angle between such straight line and the vertical on its under side (Fig. 25). This is true only when the data are plotted on the author's skew frequency paper, because otherwise the scales are apt to be inappropriate.

In all the former cases by measuring the angle the value of c is immediately obtained. That appears, also, from an analysis of the equation itself. The value of h (which depends on the units used in the data), is often easily found by reading the value of "percentage of time" at the point where the adjusted straight line crosses the horizontal line with unity as its ordinate (Fig. 25) and then applying the equation,

$$h = \left[-\log\left(1 - \frac{t}{100}\right)\right]_{f(R) = 1}$$

The factor, h, or k, depends on the system of logarithms used.

The author has devised graphical methods for accurately fixing the limits and, in some instances, has devised various mathematical formulas for the quicker determination of the various items; but normally it is found that the method of trial and error will give quickest results.

Some of those who are already expert in the use of the "normal curve" formulas and who know that in the Pearson, Charlier, and Kapteyn systems

the equation of the normal curve is a special case of the general formula, may object that the Goodrich formulas can not be reduced to the same special (Gaussian) form by the substitution of any possible group of constants. This is true, and is because the author has found it advantageous to introduce the independent variable both as a main factor as well as a factor in the exponent of e. This introduction also precludes the possibility of mathematical symmetry in the probability function. It is interesting to discover, however, that it is possible to select the constants so as to make a probability and its integral or "duration", curve that are substantially equivalent to the normal curve and its integral curve. This is shown in Fig. 26 and in Table 22.

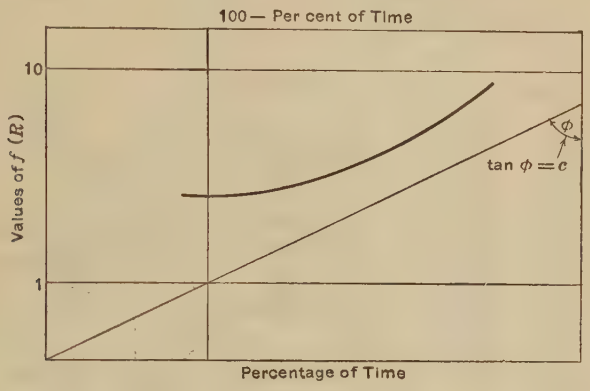

FIG. 25.

The integrals of the normal error curve will plot nearly as a straight line on the author's skew probability paper.

Again, one of the fundamental assumptions on which the normal error function is based does not hold for certain types of physical observations; that is, positive and negative errors occur in equal numbers. Clerk Maxwell points out one such instance in observing occultations.*

One great advantage of the Goodrich system of formulas is that it is possible to integrate with ease any numerical probability curve with fractional as well as integral exponents. Certain factors require the use of gamma functions when fractional exponents occur, but gamma functions are no more difficult to learn to use than logarithms.

An incidental item of interest with reference to the author's formulas is that an application of the method often used in connection with the normal curve to show that the arithmetic mean is the most probable value of all those observed, gives the same result when the author's exponent, c, is made equal to unity, and that the sum of the powers of deviations or differences is in each case a constant, varying from zero, for $c = 1$, to $\frac{1}{k}$, when $c = \infty$.

The general form of the skew frequency curve equations of Pearson, Charlier, Kapteyn, and Goodrich (using similar nomenclature as far as possible), are as follows:

* See Encyclopædia Britannica, 9th Edition, Vol. XIX, p. 781, footnote.

Pearson:

$$y = e^{\int \frac{(x+a)\,dx}{b_0 + b_1 x + b_2 x^2}}$$

Gram-Charlier:

$$y = A_0\,\phi(x) + A_3 \frac{d^3\,\phi(x)}{dx^3} + A_4 \frac{d^4\,\phi(x)}{dx^4} + \ldots$$

Kapteyn:

$$y = \frac{1}{\varepsilon\sqrt{2\pi}} \frac{d\,[F(x)]}{dx} e^{-\frac{1}{2\varepsilon^2}[F(x) - F(x_0) - M]^2}$$

Goodrich:

$$y = \frac{n}{2\varepsilon^2} \frac{d\,[F(x)]^c}{dx} e^{-\frac{1}{2\varepsilon^2}[F(x)]^c}$$

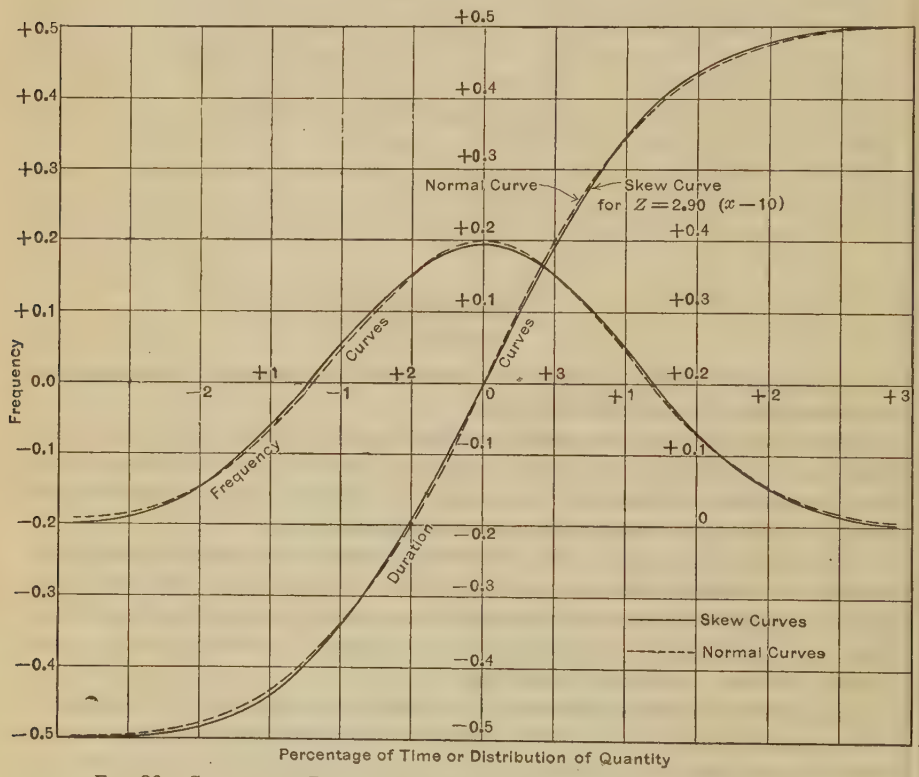

FIG. 26.—COMPARISON BETWEEN NORMAL CURVES OF ERROR AND SKEW CURVES.

In the Gram-Charlier formula, $\phi(x) = \frac{1}{\varepsilon\sqrt{2\pi}} e^{-\frac{(x-b)^2}{2\varepsilon^2}}$, so that his first term is nearly the same as the others; in the Kapteyn formula, $F(x)$ is entirely general; in the Goodrich formula, $F(x) = \frac{x-a}{(b-x)\frac{d}{c}}$.

Kapteyn builds up his formula from that of the normal form by a line of reasoning with the following conclusion*: "* * * causes independent of the size of the individuals produce normal curves, causes dependent on this size produce skew curves."

TABLE 22.—Comparison of Normal Error Curve and Special Case of Goodrich Formula.

Duration Curves.								Frequency Curves.			
Skew Curve.											
x.	$x - 10$.	$z = 0.290 (x - 10)$.	y.	$w = (y - 0.5)$.	Normal curve, $\int_0^t \phi(t)\, dt$.	Difference.	Percentage of error.	Skew curve, $w' = \frac{dw}{dz}$.	Normal curve, $\phi(t)$.	Difference.	Percentage of error.
0	−10	−2.90	0	−0.50000	−0.49813	+0.00187	0.38	0.00000	+0.00595	+0.00595	100.00
1	− 9	−2.61	0.00034	−0.49966	−0.49547	+0.00419	0.85	0.00386	0.01323	+0.00937	70.9
2	− 8	−2.32	0.00335	−0.49665	−0.48983	+0.00682	1.39	0.01910	0.02705	+0.00795	29.4
3	− 7	−2.03	0.01277	−0.48723	−0.47883	+0.00840	1.75	0.04831	0.05082	+0.00251	4.93
4	− 6	−1.74	0.03278	−0.46722	−0.45907	+0.00815	1.78	0.09204	0.08780	−0.00324	3.69
5	− 5	−1.45	0.06741	−0.43259	−0.42647	+0.00612	1.44	0.14866	0.13948	−0.00923	6.62
6	− 4	−1.16	0.11985	−0.38015	−0.37698	+0.00317	0.84	0.21387	0.20357	−0.01030	5.06
7	− 3	−0.87	0.19160	−0.30840	−0.30785	+0.00055	0.18	0.28054	0.27324	−0.00730	2.68
8	− 2	−0.58	0.28180	−0.21820	−0.22240	−0.00420	1.89	0.33940	0.33718	−0.00222	0.66
9	− 1	−0.29	0.38674	−0.11326	−0.11409	−0.00083	0.73	0.38052	0.38250	+0.00198	0.52
10	0	0	0.50000	0	0	0	0	0.39582	+0.39894	+0.00312	0.78
11	+ 1	+0.29	0.61343	+0.11343	+0.11409	+0.00066	0.58	0.38148	0.38250	+0.00102	0.27
12	+ 2	+0.58	0.71858	+0.21858	+0.22240	+0.00382	1.72	0.33960	0.33718	−0.00242	0.72
13	+ 3	+0.87	0.80849	+0.30849	+0.30785	−0.00064	0.21	0.27809	0.27324	−0.00485	1.78
14	+ 4	+1.16	0.87970	+0.37970	+0.37698	−0.00272	0.72	0.20840	0.20357	−0.00483	2.37
15	+ 5	+1.45	0.92970	+0.42970	+0.42647	−0.00323	0.76	0.14211	0.13948	−0.00268	1.92
16	+ 6	+1.74	0.96266	+0.46266	+0.45907	−0.00359	0.78	0.08763	0.08780	+0.00017	0.19
17	+ 7	+2.03	0.98203	+0.48203	+0.47883	−0.00320	0.67	0.04853	0.05082	+0.00229	4.5
18	+ 8	+2.32	0.99222	+0.49222	+0.48983	−0.00239	0.49	0.02397	0.02705	+0.00308	11.4
19	+ 9	+2.61	0.99700	+0.49700	+0.49547	−0.00153	0.31	0.01048	0.01323	+0.00275	20.8
20	+10	+2.90	0.99898	+0.49898	+0.49813	−0.00085	0.17	0.00402	+0.00595	+0.00193	30.8
Average.	....	.....	.....	.....	.....	.....	0.84	.....	.....	.....	10.00*

* This does not include the 100.00% error.

Goodrich has worked out the detailed application of an independently developed general formula of which that of Kapteyn may almost be considered a special case and in general the same basic reasoning will apply.

Kapteyn also remarks*:

"In [some] * * * cases * * * the effect of the causes of deviation cannot be independent of the dimension of the quantities observed. In such cases * * * the frequency curve will be a skew one."

In other words, if $y = f(x)$ has a normal distribution, $y = f(z)$ will have skewed distribution, where, for example, $z = x^2$, and the origin is not at the mode.

For the benefit of the novice in the use of frequency data, it is felt that an amplification of the description of the method given by the author is advantageous. First will be described in some detail the easy means of preparing and arranging data for plotting and, following this, an explanation of the methods of selecting the most suitable form of curve and of interpreting the results of the analysis. A few selected examples will be given, beginning

* See p. 13, "Skew Frequency Curves in Biology and Statistics," Kapteyn.

with the simplest and easiest form, which involves only two constants to determine the equation, and advancing step by step to the most general equation with six constants, of which one is determined directly from the data. All these examples were supplied by the author.

For the first illustration consider the data of Table 3, and Fig. 1, but use the ratio of the total run-off for each year to the mean for the eighteen years of record, instead of the percentage, as given in the table. The advantage of this will be pointed out later. These ratios are first arranged and tabulated in the order of their magnitude beginning with the lowest, as in Column (1), moving the decimal point two places to the left. When there are from about ten to forty or fifty records or observations of practically equal weight, each may be considered as the "class mark" of a series of equal percentage intervals. With n records there will then be n intervals, each covering a range of $\frac{100}{n}$ per cent. Consider the records numbered in order from 1 to n. Then for plotting each record should be assigned the percentage corresponding to the center of the interval of which it is the class mark. For any record, as the r th, the corresponding percentage will be $100\,\frac{2\,r\,-\,1}{2\,n}$. It is at once evident that the numerators of these fractions will form the series of odd numbers from 1 to $2\,n\,-\,1$. The values of these fractions can be rapidly calculated on the A and B-scales of an ordinary slide-rule with only one setting of the slide. After calculating the percentages up to 50%, the remaining values are best and most accurately obtained by subtraction, 100 minus the first value giving the last one, etc. These percentages are set down in Column (2) and the data are ready for the first trial plotting on skew frequency paper.

With the data plotted as shown in Fig. 1, it is at once evident that they lie very close to a straight line which can be located by the eye with the aid of a straight-edge or a fine thread. If the points are too irregular for this method, or if greater accuracy is desired, the line may be located by the method of centroids for logarithmic plotting. Since the author's method gives all the points equal weight, the centroid method is particularly applicable.

A great many questions about the run-off of the river at this station can now be answered directly from the curve. For example, one-half the time the total run-off will lie between the values at the intersections of the curve with the 25% and 75% ordinates, that is, between about 0.8 and 1.18 of the mean. The difference (0.38) may be called the range of average run-off or, briefly, the "average range". One-half this range is termed by Yule*, the "quartile deviation" or the "semi-interquartile range".

It is evident that one-half the total area under the differential or frequency curve, will lie between these limits. Hence, if the curve were exactly symmetrical and followed the "normal law of error," this quantity would equal the standard deviation. It is therefore an excellent measure of the "dispersion" of

* See "Introduction to the Theory of Statistics," by G. Udny Yule, 4th Edition, Charles Griffin & Co., Ltd., Chapter VIII, p. 147.

the observations and its value can be taken from the curve without the trouble of computing the "root-mean-square" of the differences of the arithmetic mean and each of the observations.

Again, the 50% ordinate intersects the curve at a height of about 0.99 of the mean run-off. Therefore, one-half the total number of observations lie on each side of this point, which is called the "median". With these three readings taken directly from the curve, one can obtain a good idea of the amount and character of the skewness of the curve. Yule gives one definition of "skewness" as the range from the median (50%) to the upper quartile (75%) minus that from the lower quartile (25%) to the median divided by the semi-interquartile range. This may be called the "quartile ratio" to distinguish it from other definitions for the term, "skewness". For this curve, the quartile ratio (Q. R.) is:

$$Q\,.\,R\,.\,=\frac{(1.18-0.99)-(0.99-0.80)}{\frac{1}{2}(1.18-0.80)}=0.0$$

Hence, this curve is approximately symmetrical, with the mean very nearly equal to the mode. Years of total run-off greater than the mean and years less than the mean are therefore about equally probable, which is an important point in any study of annual run-off for purposes of water supply. The upper end of the curve indicates that a run-off less than 1.6 to 1.7 times the mean may be expected 99 to 99.5% of the time; or to put it the other way, once in from 100 to 200 years the run-off will probably be greater than the figures stated. On the other hand the lower end of the curve shows that values less than from 0.34 down to about 0.28 of the mean are probable once in 100 to 200 years.

This one curve serves to investigate the probability of both unusually high and extremely low run-offs with a minimum of time and labor and with sufficient clearness and certainty to fill the requirements of the average engineer who is seldom interested in frequencies beyond the limits mentioned. The author states that the original purpose of the development of this graphical method of examining skew frequency data, was to make possible a "linear" extension of the "total frequency" or "duration" curve for a reasonable distance beyond the actual range of observations, and this has been done in Fig. 1. This curve is so nearly symmetrical that a similar plotting on Hazen's probability paper would doubtless lead to nearly identical conclusions.

If still further extended below the lower data, the curve reaches to zero while at the upper end it would theoretically extend to infinity, with infinitesimal frequencies long before it reaches either limit. Some one may object to the infinite limit, but this should not be sufficient cause for the rejection of the entire method in the absence of more numerous and accurate data to give conclusive evidence as to the character of the curve when extended in this direction. Conclusions based on this curve will be as safe as it is possible to obtain within reasonable limits from the data available. While the question of limits will always afford a fruitful and endless subject for discussion, it will be shown later that other of these methods of plotting fre-

quency data provide for a finite upper, as well as lower, limit, whenever the character of the data indicates such limits. In fact, it is impossible to plot the data so as to approximate a straight line in many cases, unless a finite upper limit is used in computing the linear function, which will be discussed later.

In addition to the foregoing questions, it is possible to answer many others directly from the curve. For example, suppose one wishes to know the probability of the occurrence of about 0.8 of the mean, interpreting this as a run-off from 0.75 to 0.85 of the annual average. For the lower of these figures, the curve (Fig. 1) intersects the 20% ordinate and for the upper, the 30% ordinate. Hence, the probability is the difference, or 10%; that is, a run-off of about 0.8 the average will probably occur about once every 10 years. In the theory of probabilities, areas under the differential curve represent probabilities, which therefore can be determined from the difference of corresponding ordinates to the integral curve that has been plotted; hence the great convenience in using the curve instead of the frequency curve.

If the arithmetical magnitude of the "most probable value" of the run-off is desired, it is first necessary to determine the values of the constants in the equation of the curve, which takes the form:

$$t = 100\,(1 - 10^{-h\,R^c}) \qquad (S)$$

in which, R is the ratio of any value of the annual run-off to the average, and t is the corresponding percentage of the total time. Also, the index, c, is the slope of the line drawn through the points with respect to the axis of run-off while the modulus, h, is the intercept. In this case its value is $-\log\left(1 - \frac{t}{100}\right)$ for the percentage of time at the intersection of the curve and the horizontal line at the value of R equal to unity. If the percentages with respect to the mean run-off are used instead of the ratio to the mean, the intercept would have fallen outside the limits of the diagram, as in Fig. 1. The values of the index and modulus are easily taken from the curve and the resulting equation is, therefore:

$$t = 100\,(1 - 10^{-0.3\,R^{3.85}})$$

or,

$$t = 100\,(1 - e^{-0.69\,R^{3.85}})$$

The most probable value of the run-off is now obtained from the following formula for the "mode" which indicates the location of the maximum point on the frequency curve and the point of inflection on the duration curve when they are plotted with ordinary rectangular co-ordinates:

$$\text{Mode} = \left(\frac{c - 1}{c\,k}\right)^{\frac{1}{c}} = 1.01$$

in which, $k = \frac{h}{M}$ is the modulus of the equation when the base, e, of the natural system, instead of the base of the common system, is used.

The foregoing example illustrates the simplest and easiest conditions to analyze graphically, but often the data will follow a decided curve when plotted as described. Examples of such curves are shown in Figs. 2 to 6 of the paper. While the duration curves in Figs. 2, 3, 4, and 6 are all more or less concave upward, they illustrate widely differing forms of frequency curves. All of them are decidedly unsymmetrical. In Fig. 2 the mode is slightly less than the mean, while in Fig. 3 it is only about two-thirds of the mean, giving very large positive skewness. In Fig. 6 the curve has the special one-sided form of maximum positive skewness, while in Fig. 4 the mode is greater than the mean, producing so-called negative skewness. In these four cases the curvature indicates a lower limit greater than zero. When suitable values for these limits are subtracted from the observations and they are replotted they follow the dotted lines as shown. Hence, the required function for linear plotting for these examples is $(R - a)$.

In Fig. 5, the first plotting of the data is concave downward. This indicates that the theoretical frequency curve extends beyond zero to a negative limit. When the value of this limit is added to the original observations, the points can be plotted so as to approximate closely the auxiliary curve as drawn. As noted in the paper, the practical interpretation of this geometrical condition is that, in the case of this river, seasons of no run-off appear to be probable about 1.5% of the time, or about once in 65 to 70 years, and not that negative discharge is possible.

A final illustration of this type of curve with the lower limit greater than zero, but with practically a maximum of skewness, will be given. Fisher* describes this as a "classical illustration" from the works of Professor Karl Pearson and uses it to illustrate the application of the very elaborate method of the "Gram-Charlier series" for the fitting of a curve to a very skewed frequency distribution by using "semi-invariants" and least squares, a method previously thought by some to be impossible in such cases. The problem may be solved graphically with the greatest ease and in a very short time, and the curve gives results directly which agree as well as those of Fisher and are much closer than those of Pearson. Furthermore, it is not even necessary to determine the equation of the curve to do this, although that can be written by simply finding the slope and intercept. The only constant that was required for the solution was the lower limit of the curve, and this was evident from the data themselves. Another reason for the use of this illustration was to show the method of preparing the data for plotting when they are given as a frequency distribution arranged from the statistics obtained from hundreds or thousands of observations.

The first and second columns of Table 23 give the number of petals and the number of flowers observed with the given numbers of petals. The total number of flowers examined (222) is the sum of the items in Column (2) and from this it is easy to compute the percentage frequencies given in Column (3). The progressive sums of these frequencies gives the total percentage frequencies in Column (4) which are used with the quantities in Column (1) for the trial plotting shown by the dots in Fig. 27. It is evident that a

* "Mathematical Theory of Probabilities," by Arne Fisher.

straight line will not fit these data, which follow a smooth curve concave upward. The data show that none of the flowers had as few as four petals although more than one-half the total number had five. The trial value for the lower limit, therefore, was made 4, and this was subtracted from the quantities in Column (1) and the differences set down in Column (5). These items were then plotted against the corresponding percentages in Column (4) as shown by the small circles in Fig. 27. The dotted auxiliary line was then drawn through the two lowest points, which should have the greatest weight, as the flowers with these numbers of petals had the greatest frequencies. It is evident that the line fits the data very well and a smooth curve can now be drawn through the original points as fully described in the paper, and indicated in Fig. 27, by plotting a series of points four units above the auxiliary curve.

TABLE 23.—Petal Flowers in Ranunculus Bulbosus.*

Number of petals in flower, x.	Number of flowers observed.	Percentage frequency.	Total percentage frequency, $\frac{100\,f}{n}$.	$x - 4$.	Total percentage frequency.	Total frequency.	Computed frequency.	Percentage of error.	Fisher.		Pearson.	
									Computed frequency.	Percentage of error.	Computed frequency.	Percentage of error.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
5	133	59.9	59.9	1	60.0	133.2	133.2	− 0.2	132.6	+0.4	136.9	− 3.9
6	55	24.8	84.7	2	84.8	188.2	55.0	0.0	55.2	−0.2	48.5	+ 6.5
7	23	10.4	95.1	3	94.3	209.2	21.0	+ 2.0	22.9	+0.1	22.6	+ 0.4
8	7	3.1	98.2	4	98.0	217.5	8.3	− 1.3	7.1	−0.1	9.6	− 2.6
9	2	0.9	99.1	5	99.3	220.3	2.8	− 0.8	3.0	−1.0	3.4	− 1.4
10	2	0.9	100.0	6	99.75	221.3	1.0	+ 1.0	0.1	+1.9	0.8	+ 1.2
..	222	.....	.....	...	.....	.....	Average.	0.9	.....	0.6†	.....	2.7

* Data from the "Mathematical Theory of Probabilities," by Arne Fisher, p. 226.
† Using seven constants; for five constants the average error was 2.1 per cent.

It is interesting to compare the results obtained graphically with those obtained by the elaborate analytical methods. This can be done by multiplying the total percentage as read from the curve for each of the given numbers of petals, by the total number, n, of flowers examined, which is 222. These quantities are set down in Columns (6) and (7). The first differences of the items in Column (7) are given in Column (8) and are the theoretical frequencies which may be compared with the observed frequencies given in Column (2) and also with the results obtained analytically by Pearson and Fisher which are given in Columns (10) and (12). The errors between the observed and computed quantities with their averages, taken without regard to sign, are given in Columns (9), (11), and (13). When the labor involved in the analytical methods used by either Fisher or Pearson, is compared with the ease and clearness of this graphical solution, the latter presents very definite advantages especially in the hands of those with practical judgment and experience in the use of graphical methods of computation and analysis.

The curve (Fig. 27) shows that the probability of finding a flower with only four petals is zero, while there is only about one chance in a thousand of finding one with eleven or more petals. The graphical results have been compared with those obtained analytically by two different methods, both of which require the determination of three or more constants, while only two were used here, and both of these were obtained by inspection of the data. The first was the number of observations, n, while the second was the lower limit, a, of the frequency curve. (Fisher's very close results required the determination of seven constants. With five constants, his results are little

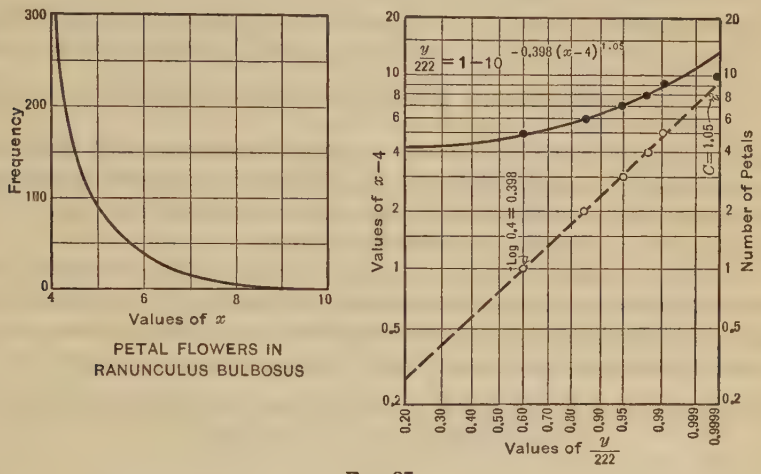

FIG. 27.

better than those of Pearson, the average error being 2.1 per cent.) As previously stated, it is only necessary to take off the slope and intercept in order to obtain two additional constants and then to write down the equation of the curve. It is then possible to compute any of the theoretical values of any of the usual measures used for purposes of comparison with those obtained from the data or from other frequency series. The equations of this curve referred to the two ordinary systems are:

$$y = 222\ (1 - 10^{-0.398\ (x-4)^{1.05}})$$

and,

$$y = 222\ (1 - e^{-0.916\ (x-4)^{1.05}})$$

The formulas for the constants and coefficients referred to as the usual measures of comparison, based on the equation with the lower limit, a, are given by the author's Equations (1) and (5) to (9), inclusive.

The next step in the generalization of the equation for these frequency curves, was to provide for the many cases when the data, as plotted on logarithmic or skew frequency paper, showed a reversal in the direction of curvature, pointing to or suggesting a finite upper limit. After trials with various combinations it was found that the purpose could be accomplished by putting the equation in the following forms:

$$t = n\left(1 - 10^{-h\left(\frac{R-a}{b-R}\right)^c}\right)$$

or,

$$t = n\left(1 - e^{-k\left(\frac{R-a}{b-R}\right)^c}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

In these equations, b is the finite upper limit while the other constants and variables are the same as before. One advantage in this system of frequency curves is that the constants have the same significance in all the equations whatever the limits and for all values of skewness. This is not the case with the Pearson curves for which the equations take widely differing forms for different limits and skewness.

Data showing this reversal of curvature are illustrated in Figs. 7 and 8. It is somewhat more difficult to fit curves to such data, since it is necessary to determine suitable values for both limits, a and b, before the linear function, $\left(\frac{R-a}{b-R}\right)$, can be plotted. While it is possible theoretically to obtain approximate values of these constants by graphical methods with data taken from preliminary frequency and duration curves, it is usually quicker and easier to find them by trial unless the data are so numerous and regular as to enable one to draw very smooth preliminary curves.

With a little practice it will be possible in most ordinary cases to select suitable values for the limits after a few trials and when these have been determined, small changes in the lower limit will alter the alignment at the lower end of the curve much more than at the upper, and *vice versa.*

TABLE 24.—Annual Maximum Flood Heights on the Sze Ya Ho, at Ti Liu Pu, North China, 1912-1925.

Maximum flood level, in meters, G. (1)	Percentage of the total time, t. (2)	$G - 2$. (3)	$10 - G$. (4)	$\frac{G-2}{10-G}$ (5)
3.2	3.6	1.2	6.8	0.18
4.6	10.7	2.6	5.4	0.48
4.8	17.8	2.8	5.2	0.54
4.9	25.0	2.9	5.1	0.57
5.6	32.1	3.6	4.4	0.82
5.6	39.3	3.6	4.4	0.82
6.3	46.4	4.3	3.7	1.16
6.6	54.5	4.6	3.4	1.35
6.6	60.7	4.6	3.4	1.35
7.0	67.9	5.0	3.0	1.67
7.6	75.0	5.6	2.4	2.33
7.6	82.2	5.6	2.4	2.33
7.8	89.3	5.8	2.2	2.46
8.0	96.4	6.0	2.0	3.00

In addition to the illustrations in the paper, the data in Table 24 afford a characteristic example of this same type. This table gives the annual maximum flood heights from 1912 to 1925, inclusive, on the Sze Ya Ho, at Ti Liu Pu, near the center of the Chihli Plain, in North China. In this table, G is the elevation of the maximum water level, in meters, above Taku Datum, as given in Column (1), with the corresponding percentages of the time in Column (2). The limits, 2 and 10, were used to compute the other quantities in Table 24 after three other combinations were also tried with limits of 1 and 9. The auxiliary curve (Fig. 28) can be located by eye so as

to pass through one-half these points with the others scattered on both sides of the line, giving very satisfactory results. The curve shows that one-half the maximum flood heights are between Elevations 5.25 and 7.25, giving an average range of just 2 m. One-half the floods are higher than 6.4 m., and hence the skew ratio is — 0.30. As this is negative, the mode is greater than the mean, indicating that flood heights above the mean are more probable than those below.

Dangerous stages occur when floods rise higher than an elevation of about 7.5, which the curve shows to be probably about 16% of the time, or once in 6 or 7 years. Two such floods have occurred during the period of observation, once in 1917 when the City of Tientsin was greatly damaged and again in 1924 when the city was threatened for several weeks with a similar flood. Tientsin was flooded in a similar disastrous manner about thirty years prior to 1917, and it was generally thought that dangerous floods were probable only about once in that period, but investigations show that there might be as many as five or six dangerous floods on the plains west of the city during that period. The curve also shows that a flood as high as 8.5 m. above Taku Datum is not probable more than once in 100 years, while floods only ½ m. lower may be as frequent as once in 15 years.

If a straight line had been drawn through the points as originally plotted, the lower and middle portions of the curve would have been effected but little. If, however, this line were extended to estimate the probable occurrence of floods and their height from 1 to 10% of the time, very erroneous conclusions might have been reached, such as that a flood height of 9.0 m., or more, might be expected every 50 years. As the present river dikes have never been raised higher than Elevation 8.0, Taku Datum, they would all be overtopped or breached long before such an extreme elevation could be reached. They have been in existence for more than 100 years. Attention is called to these facts in order to emphasize the necessity of taking physical conditions into account even when dealing mathematically with such problems.

The "most probable" flood height is determined by the mode, for which the modulus and index must first be determined from the auxiliary curve (Fig. 28). The intercept is at the 40% ordinate and is, therefore:

$$-\log.\ (1 - 0.4) = 0.222 \text{ nearly.}$$

The slope of the auxiliary curve is 1.52 and hence the equation of the curve is:

$$\frac{t}{100} = 1 - 10^{-0.222\left(\frac{G-2}{10-G}\right)^{1.52}}$$

A direct solution for the mode cannot be made for this equation, which involves two finite limits, but an approximate value can be determined by repeated trials, by substitution in the following equation:

$$(c - 1)(b - R) + (c + 1)(R - a) = \frac{c\,h}{M}(b - a)\left(\frac{R - a}{b - R}\right)$$

This equation can be simplified by putting the ratio, $\frac{(R - a)}{(b - a)}$, equal to z and solving for h. Thus:

$$h = M\left(\frac{1 - z}{z}\right)^{c}\left(1 + \frac{2z - 1}{c}\right)$$

A set of curves may be plotted from this equation. If the base, e, is used instead of the base, 10, it is only necessary to remember the relation, $k = h\,M$, and make the necessary substitutions. Entering such a diagram with corresponding values of h and c from the equation of the curve, an approximate value of z could be found. From this the mode may be determined by the formula:

$$\text{Mode} = z\,(b - a) + a$$

For the foregoing example, the mode is approximately 6.68 m. which is, therefore, the "most probable" flood height sought.

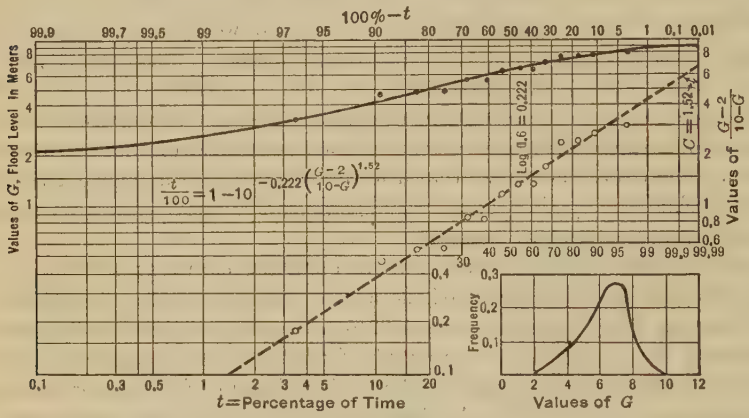
100%−t
99,9 99,7 99,5 99 97 95 90 80 70 60 50 40 30 20 10 5 1 0,1 0,01
Values of G, Flood Level in Meters
Values of (G−2)/(10−G)
C = 1,52
Log 0,6 = 0,222
t/100 = 1 − 10^−0,222((G−2)/(10−G))^1,52
Frequency
t=Percentage of Time
Values of G

Fig. 28.—Annual Maximum Flood Heights on Sze Ya Ho, at Ti Liu Pu, China.

Although the special equations just given cover the majority of the more common frequency curves, data will sometimes be found which can not be adequately represented by these curves even when four constants are used, in addition to n, which is usually determined at once by the data. However, there are still other very similar special equations derived from the same general frequency equation, which can be used in the same way on this skew frequency paper.

The general form of this empirical equation for skew frequency curves, using the base, e, is for the differential curve:

$$t' = n\,k\,[c\,(b - R) + d\,(R - a)]\left(\frac{R - a}{b - R}\right)^{\frac{c-1}{d+1}}\left[e^{-k\frac{(R-a)^c}{(b-R)^d}}\right] \ldots . (G')$$

This equation, the author states, was obtained by differentiation after the fundamental equation of the integral curve had been developed by successive stages of generalization.

The special equations already discussed are easily obtained from Equation (G') if both c and d are considered as positive, or when one index is zero and the other is positive; but the integration can also be made when these exponents are considered negative. This amounts to inverting the fraction, $\frac{(R - a)}{(b - R)}$, and does not change the signs of these two constants in the first term in

brackets where both c and d are positive. Integration gives the three special equations a slightly different form in this case, in which the constant term, n, becomes zero. They are:

$$t = n\, e^{-k\, R^{-c}} \quad \ldots\ldots (T)$$

$$t = n\, e^{-k\, (R-a)^{-c}} \quad \ldots\ldots (D)$$

$$t = n\, e^{-k\, \frac{(R-a)^{-c}}{(b-R)^{-d}}} \quad \ldots\ldots (E)$$

One or two illustrations will show when these equations should be used and how the data are handled. First, take an example from Yule (Table 25) which gives the mean annual death rate per 1 000 of population in certain districts in England.

TABLE 25.

Mean annual death rate per 1 000, R.	Number of districts with given death rate.	Frequency, percentage.	Total frequency, t, percentage.
12.5–13.5	5	0.79	0.79
13.5–14.5	16	2.53	3.32
14.5–15.5	61	9.65	12.97
15.5–16.5	112	17.73	30.70
16.5–17.5	159	25.13	55.83
17.5–18.5	104	16.45	72.28
18.5–19.5	67	10.60	82.88
19.5–20.5	42	6.65	89.53
20.5–21.5	25	3.96	93.49
21.5–22.5	18	2.85	96.34
22.5–23.5	8	1.27	97.61
23.5–24.5	5	0.79	98.40
24.5–25.5	3	0.48	98.88
25.5–26.5	1	0.16	99.04
26.5–27.5	1	0.16	99.20
27.5–28.5	2	0.32	99.52
28.5–29.5	....	.....	.....
29.5–30.5	....	.....	.....
30.5–31.5	2	0.32	99.84
31.5–32.5	....	.....	.....
32.5–33.5	1	0.16	100.00

When these data are plotted in the usual manner (Fig. 29), rising from left to right, the lower end is extremely flat while the points at the upper end fall very close together and rise too rapidly to be fitted by any of the curves previously used. The percentage ordinates are spaced much more widely at the left side of the diagram so that if the upper points were plotted at that side the curve would not bend upward so sharply. Accordingly, the data are replotted rising from right to left as shown in the upper curve of Fig. 28, using the percentage scale at the top of the diagram. A line drawn through the third and thirteenth points fits the intervening ones with remarkable exactness while departing but little from those at either end. The slope of this line is evidently negative as compared with the previous diagrams, which indicates that Equation (T) will fit the case. As the intercept of this curve is outside the limits of the diagram, the values of the constants for the equation can best be determined by substituting the co-ordinates of two points on the curve in the equation written in the logarithmic form, and solving for the unknown modulus and index. The resulting equation is:

$$t = 100\ (10)^{-319\ (10)^{10}\ R^{-10.55}}$$

in which, t is the percentage of the total number of districts having the death rate, R. The number of districts, 632, may be substituted for 100 in the equation, and t will then give the approximate number of districts instead of the percentage. The most frequent rate will be given by the mode, but the formula for this as well as for the standard deviation, etc., differs slightly from those given previously. For the mode and mean, the formulas for Equation (D) are:

$$\text{Mode} = a + \left(\frac{c\,k}{c+1}\right)^{\frac{1}{c}}$$

$$\text{Mean} = a + k^{\frac{1}{c}}\,\Gamma\left(1 + \frac{1}{c}\right)$$

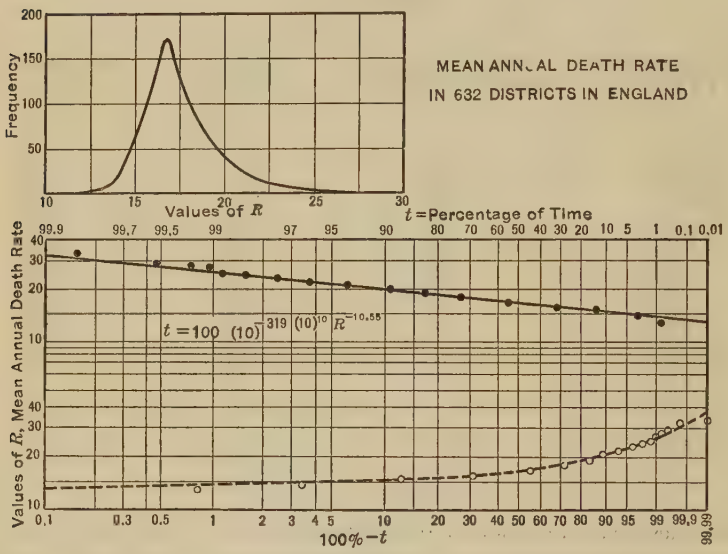
MEAN ANNUAL DEATH RATE
IN 632 DISTRICTS IN ENGLAND
Frequency
Values of R
t = Percentage of Time
Values of R, Mean Annual Death Rate
t = 100 (10)^-319 (10)^10 R^-10.55
100% − t

Fig. 29.

Other equations can be written by analogy, but all the formulas which involve the gamma function fail for values of c that produce negative integral values of gamma because the gamma function then becomes infinite. Fractional negative values give finite values of the function.

Another illustration is that of the Wimmera River at Glenorchy, Australia, shown in Fig. 11, where the curve is plotted rising from right to left. Even the use of Equation (C) with four constants does not give an entirely satisfactory curve, while plotting the data in the other direction is out of the question. By using the general Equation (F) with five constants, in which, c and d are negative, a very good curve (considering the irregularity of the data) can be obtained, as shown by the dotted curve in Fig. 11. The solution is made in the following manner: The limits are first assumed at values which appear reasonable from the original data and values of $f(R)$ are computed with a slide-rule as usual. The computed ratios are then plotted and

are found to follow a slight curve instead of a straight line. The general equation, when written in the logarithmic form, is:

$$\log\left(-\log\frac{t}{n}\right) = \log h + c\log\,(R-a) - d\log\,(b-R)$$

The co-ordinates of three points well distributed along the auxiliary curve, are chosen and substituted in this equation. The simultaneous equations thus formed are then solved for the values of the exponents, c and d, from which the ratio, $\frac{d}{c}$, is determined. $f\,(R)$ may be written:

$$\frac{(R-a)^c}{(b-R)^d} = \left[\frac{R-a}{(b-R)^{\frac{d}{c}}}\right]^c$$

Values of $(b-R)^{\frac{d}{c}}$ can now be computed, together with values of the linear function within the brackets in the right-hand member of the last equality. When these are plotted they will be found to follow a straight line much more closely and a new auxiliary curve can now be drawn. From this curve corrected values for the exponents and the modulus can be found as usual. In the equation for the final dotted curve of Fig. 11, it will be seen that the ratio, $\frac{d}{c}$, is $\frac{1}{3}$. Usually some such common ratio can be used to simplify the computations so as to use a slide-rule. In other cases a "log log" rule is very convenient.

The data for the annual run-off below the dam on the Sweetwater River, California, give an especially interesting curve. (See Table 11 and Fig 9.) Here, the lower limit of the curve is negative since years of no discharge are quite frequent. The plus (+) sign in the denominator of the linear function gives an upper limit of infinity instead of a finite one. The interpretation of the frequency curve is particularly interesting and suggestive. The storage of water in the reservoir and the fact that it is used for irrigation through canals instead of being drawn off in the river bed, causes an accumulation of points near the lower limit. On the other hand, the reaction of this river to the causes producing years of large total run-off, seems to increase very rapidly with an increase in the intensity of those causes, as indicated by the long, nearly flat, right-hand portion of the frequency curve (Fig. 9) from 100 to 1 000% of the mean. Special attention is called to the three principal characteristics of this curve, as they are fully explained, in two instances at least, by particular physical conditions very evidently causing the finite frequency of zero run-off and the large number of years of small discharge due to the storage and use of water for irrigation. The causes producing the wide spread in the frequency curve for the years of large discharge are no less real although more obscure in the absence of complete information.

One of the most valuable features in frequency studies is the fact that, with reliable data, any departure from the ordinary shape or character of a frequency curve, points to some particular cause or causes which a careful

study of conditions should disclose, thus making possible a more intelligent solution of the problem in hand.

For example, a report was made by Sigurd Eliassen, Assoc. M. Am. Soc. C. E., on the loess and silt carried by the rivers of North China. Frequency curves were shown for the percentage (by volume) of silt of varying diameters, as determined from a microscopical study of samples taken at three different stations of the Yung Ting Ho. All these curves were plotted with ordinary rectangular co-ordinates. The curve for the silt from the lowest station differed so widely from those for the two other stations that a question was raised as to the accuracy of all the curves. An analysis made on skew frequency paper, however, disclosed remarkable consistency as to the peculiar character of the distribution of the medium and larger diameters, while the comparatively low percentage of the smaller diameters in the curve for the lowest station was accounted for by the fact that those samples were taken from the river bed. At the upper stations all the samples had been taken from the stream during a high flood. Thus, they contained a certain quantity of very fine material which would not have a chance to settle and would be carried past the lower station. As a result, confidence in the curves was increased as they were shown to be fundamentally consistent.

Another and most unusual curve is shown in Fig. 17. While the data themselves are of less interest to engineers, the curves are enlightening as the distribution is practically the opposite of that just described. For this curve, Equation (*C*) is used, which has minus infinity as the lower limit with a finite quantity as an upper limit. The frequency curve shows a considerable number of specimens of flax having zero length and a long flat curve for the frequencies of the shorter lengths of stalks with a large accumulation of items near the upper limit. Kapteyn's comments on this subject also show how the frequency curve can be interpreted in terms of the physical conditions surrounding the growth of the specimens under investigation.

Another application of this graphical method of analysis is suggested by the possibility of resolving certain frequency distributions which show some peculiar irregularities into two (or more) simple and regular curves. The average annual rainfall at San Francisco and Sacramento, Calif., presents an interesting example of this kind. In the paper by C. E. Grunsky, Past-President, Am. Soc. C. E., on "Rainfall and Run-Off Studies",* the curve drawn through the points in his Fig. 4,† does not indicate any unusual characteristics. The records as combined in his Table 2,‡ cover a period of more than seventy years, and it is possible to draw a smooth curve through the points which averages them very well. When the data are plotted on skew frequency paper, however, as shown by the dots along the lower curve in Fig. 30, they follow a line that is approximately straight almost to the 90% ordinate where it has a pronounced upward trend, after which it again becomes nearly parallel to the lower portion. (On account of the large number and closeness of the points along the middle of the curve, only alternate

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 66.
† *Loc. cit.*, p. 79.
‡ *Loc. cit.*, p. 77.

ones are plotted from the 20% to the 85% time ordinates.) The points, 10% and 240% of the mean annual precipitation, were selected as the probable limits, and values of the linear function, $\frac{(R - 10)}{(240 - R)}$, were computed and plotted as shown by the circles along the upper curve. After numerous trials the two auxiliary curves (shown by the dashed lines, Fig. 30) were drawn to represent two independent frequency distributions. They have the same abso-

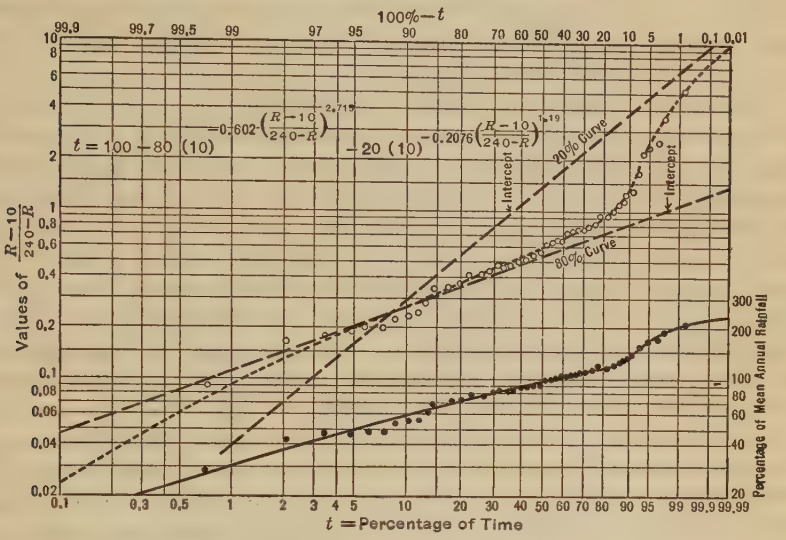

FIG. 30.—AVERAGE ANNUAL RAINFALL AT SAN FRANCISCO AND SACRAMENTO, CALIF.

lute limits, but quite different ranges within the practical limits of the diagram. The dotted auxiliary curve is formed from the two component curves by adding along horizontal lines for any given ratio, $\frac{(R - a)}{(b - R)}$, one-fifth of the reading for the line with the greater slope and four-fifths of that for the line with the lesser slope, to give the percentage of the total time for the corresponding point on the dotted curve. Thus, for a value of the ratio of 0.9:

$$\frac{1}{5} \times 34\% + \frac{4}{5} \times 93\% = 6.8\% + 74.4\%$$

or 81.2%, which gives the required point on the auxiliary curve for this value of the ratio. A series of points were found in this way and the dotted auxiliary curve was drawn, after which it was an easy matter to plot the curve through the original data in the lower part of Fig. 30.

From this analysis one is led to the tentative conclusion that, on the average, about 20% of the total annual rainfall is due to storms of a particular kind or class (which includes most of the heavier rains), while the ordinary precipitation, amounting to about 80% of the total, is of another and inde-

pendent class or classes, probably differing materially as to origin or direction, or in some other way. The mean annual rainfall in Hawaii shows very similar characteristics.

A like conclusion has been strikingly confirmed within the past few months in connection with studies for the purpose of flood control on the Yung Ting Ho, one of the larger rivers in the Province of Chihli. This river has its head-waters and the greater part of its basin on the high plateau north and west of the Great Wall, beyond the mountain range which crosses the Province. The river passes this range through a long canyon and enters the plain near the capital, Peking, thence flowing across the plains before joining other rivers near the Port of Tientsin.

In making a graphical analysis of the gauge heights at the station nearest the mountains, the resulting frequency curve that most closely approximated the data, was a combination of two frequency curves very similar to those just described (Fig. 30). No single curve could be found even to approximate the requirements, although many trials were made previous to the resolution into two component curves. The conclusion thus indicated that two independent sets of causes producing run-off combine to bring about the distribution of river stages shown by the records, was confirmed by an independent investigation covering several years. This was made by Mr. Eliassen who has shown conclusively that rains in the northern part of China during the summer flood season are from two different sources. One class are cyclonic storms originating in areas of low barometric pressure, which form over Central and Western Mongolia and move in a southeasterly circle toward Japan. Other storms are of a local orographic character which develop along the coast and move in a northerly direction toward the mountains, while the most severe storms may also be classed as coastal, although they are caused by typhoons, which usually form in the South Pacific and move northward along the coast of China. These typhoons sometimes pass over parts of North China west of the Shantung Peninsula, although they usually pass to the east of the peninsula toward Japan. The radical difference in these storm sources is evident, and is accurately reflected in an analysis of the resulting phenomena of river stages.

Although skew frequency data may be plotted by the use of these equations with ordinary rectangular co-ordinates, or on logarithmic paper, it is more quickly and easily done on the author's skew frequency paper. The curves plotted on any of these three kinds of paper will have exactly the same shape if the same scales are used in both directions so that the interpretation of the curves is the same; but it is much easier with the special paper.

While certain population records can be quite closely approximated with these equations, as shown by the examples given in the paper, the difficulty in the determination of the value of n, which is the limiting population, is such as to render the equations of doubtful value for this purpose.

In Figs. 31, 32, and 33, are two series of curves prepared to show the changes in shape produced by variations in the values of c and of h. Fig. 34 shows the shape of typical complete curves drawn from the author's formula,

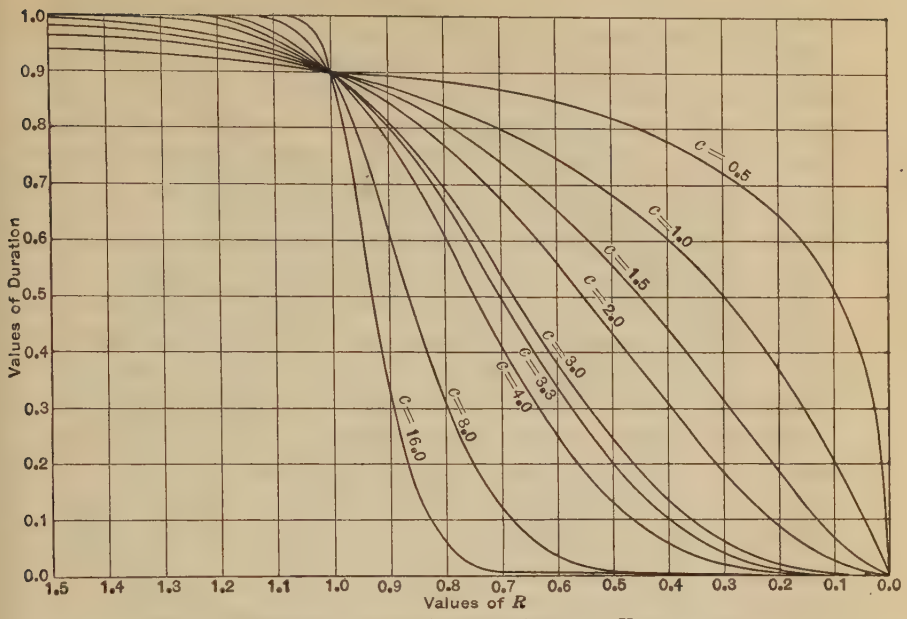

FIG. 31.—DURATION CURVES SHOWING EFFECT OF VARIATION IN c.

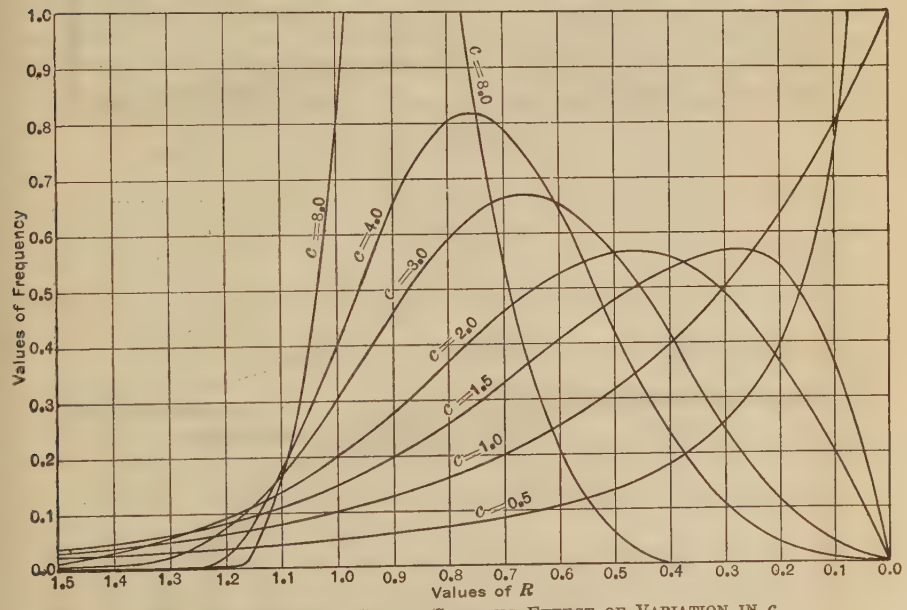

FIG. 32.—FREQUENCY CURVES SHOWING EFFECT OF VARIATION IN c.

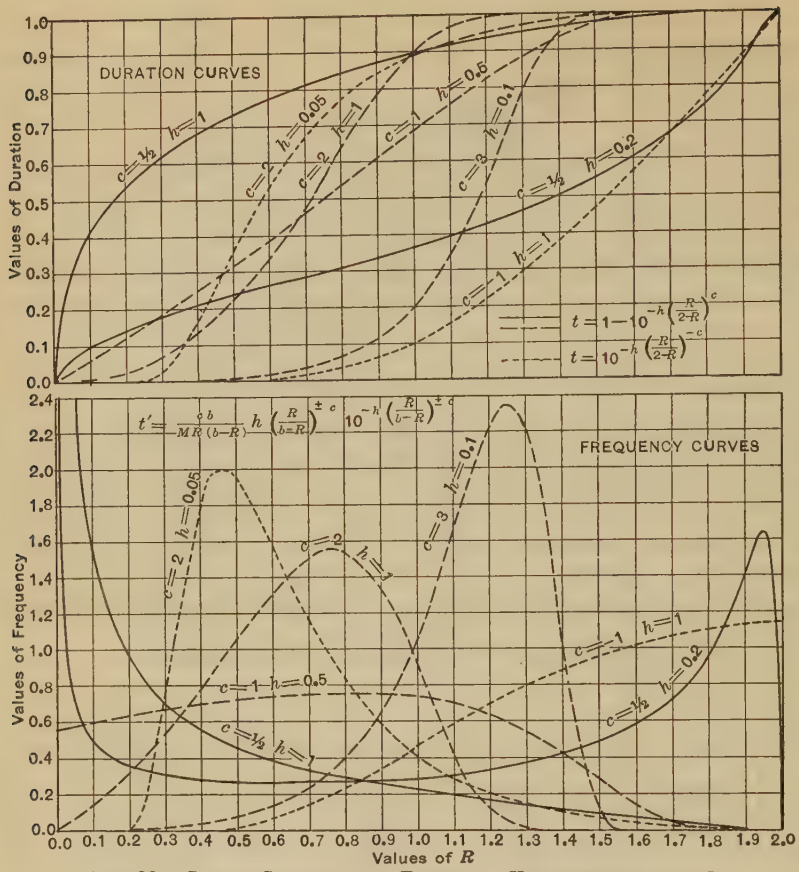

FIG. 33.—CURVES SHOWING THE EFFECT OF VARIATION IN c AND h.

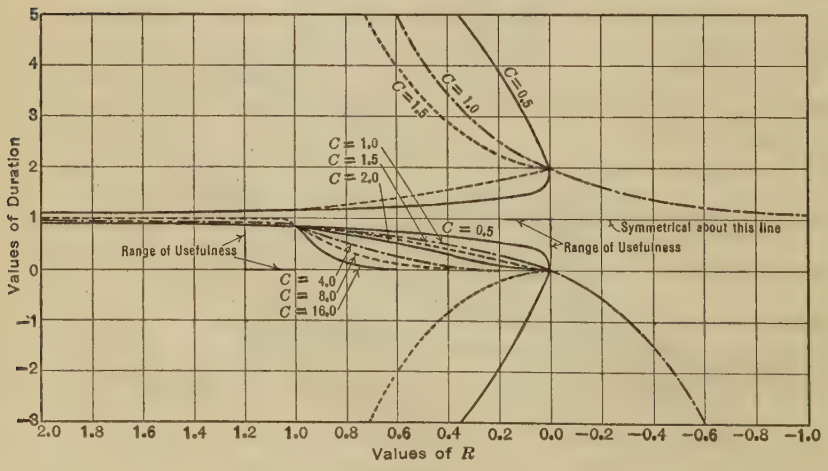

FIG. 34.

together with the limits within which the formula is used. These extended curves are obviously only of academic interest.

In the writer's discussion* of the paper by Allen Hazen, M. Am. Soc. C. E., entitled "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply", the hope was expressed that some one might develop a frequency formula in which the several constants would have special significances. It appears that the author has largely accomplished this ideal.

It is believed that this question of the theory of probabilities as applied to engineering investigations is a subject on which more engineers in the future will become interested than have been in the past. It is one that is being used by all classes of technicians to-day, and it is to be hoped that the author's method will be tried by those who have been accustomed to other methods, that its weak points will be disclosed, and, if it is of value, that it will be as widely used as it is believed that its actual merits deserve.

Arne Fisher,† Esq. (by letter).—Reference has been made during the discussion to the writer's citation regarding the distribution of flower petals among specimens of *Ranunculus Bulbosus* as collected by the Dutch botanist, De Vries. This particular reference may perhaps create an erroneous impression among certain readers. As specifically mentioned‡ this illustration and its subsequent treatment by the method of least squares in conjunction with the Gram series was merely chosen as a drill in the formation and subsequent solution of the normal equations, while in the actual practice of curve fitting, resort would be had to a logarithmic transformation, or to the Poisson-Charlier series. This latter method of finding the frequency curve of flower petals§ can be performed in less than 15 min. As a matter of fact, the complete details of all the calculations are as given in Table 26.

Values for the different quantities will then be as follows:

$$\lambda_0 = c_0 = 222;\ \lambda_1 = m = 140 : 222 = 0.631;$$
$$\lambda_2 = 292 : 222 - \lambda_1{}^2 = 1.315 - 0.398 = 0.917;$$
$$c_2 = \frac{1}{2}(\lambda_2 - m) = \frac{1}{2}(0.917 - 0.631) = 0.143;$$

and,

$$F(x) = c_0 [\phi_m(x) + c_2 \Delta^2 \phi_m(x)] = 222 [\phi_m(x) + 0.143 \Delta^2 \phi_m(x)]$$

in which, $\phi_m(x) = \dfrac{e^{-0.631}\, 0.631^x}{x!}$ is the usual Poisson probability function of the form, $e^{-m}\, m^x : x!$, for which extensive tables with first and second differences have been published for various values of m from 0.1 to 15.5. It takes but little experience with the theory of frequency curves to convince any competent investigator that this method is superior to the quasi-analytical graphical process suggested by the author.

There are two distinct uses to which the theory of frequency functions has been put, one being merely that of a purely empirical graduation process, and

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1646.
† Actuary, Western Union Telegraph Co., New York, N. Y.
‡ "The Mathematical Theory of Probabilities," by Arne Fisher, p. 226.
§ *Loc. cit.*, p. 274.

the other, that of arriving at the probable or presumptive values of the statistical parameters of future observations. The paper deals only with the subject of graduation and leaves almost untouched the far more subtle and difficult aspects of the probable or presumptive values of future observations.

TABLE 26.—Schedule for Computing the Poisson-Charlier Frequency Curve for the Distribution of Petal Flowers in Ranunculus Bulbosus.

Number of petals.	x.	Observed frequency, $O(x)$.	$x\,O(x)$.	$x^2\,O(x)$.	Poisson-Charlier, frequency curve, $F(x)$.
5	0	133	0	0	134.5
6	1	55	55	55	51.6
7	2	23	46	92	22.5
8	3	7	21	63	9.5
9	4	2	8	32	2.9
10	5	2	10	50	0.6
..	..	222	140	292	221.6

The author states most frankly that his equations are "entirely empirical". His problem seems to be that of finding a relatively simple formula for the integral of a generalized frequency curve, and he suggests as a solution to this problem the following formula:

$$t = m - n\,(10)^{-h\frac{(x-a)^c}{(b-x)^d}}$$

This equation appears to be a special case, although in a greatly modified form, of Quiquet's general formula for the "integral" (or "duration") curve for the number of survivors, $l(x) = t$, in a mortality table. Quiquet writes his generalized formula for the duration curve in the form:

$$\log l(x) = A + Bx + \sum e^{k_i x} f_i(x)$$

in which, the k's, are certain constants and $f_i(x)$ polynomials possessing some important properties which have been discussed in detail.*

As pointed out by the author the Dutch mathematicians, Kapteyn and Van Uven, have also suggested similar forms of equations in their two well-known papers on "Skew Frequency Curves in Biology and Statistics". For the benefit of readers interested in the subject it may be stated that two later publications by Van Uven, dealing with skew frequency curves and logarithmic frequency distributions, were published during 1916.†

While Van Uven's method is based on a sound and rigorous theoretical foundation (namely, that of finding the function, $h(x)$, of the variable, x, which will be normally distributed, although the variable, x, itself is not normally distributed), the author's purely empirically derived formulas appear to be nothing more than very neat graduation formulas. As such they are most valuable if one is satisfied with graduating certain observations by the

* *Bulletin trimestriel* des Actuaries Français, 1893.

† Koninklijke Akademie van Wetenschappen te Amsterdam, Vol. 19 (1916).

particular graphical process described in the paper; but as probability formulas for presumptive frequency curves and as a basis for the forecast of future events or observations, they seem open to various objections.

The several constants in the author's formula are, like all empirically derived values, subject to sampling errors and it seems that one of the paramount requirements would be to furnish reliable formulas for determining the standard (or probable) errors of these constants. For instance, if it should be found that a constant has the numerical value of, say, 0.0345, with a standard error of ± 0.105, there are evidently no good reasons for attaching any importance to the constant itself.

The author does not furnish any formulas for the determination of such standard errors, and the writer has not been able thus far to work out such formulas himself. On the other hand, all the numerical examples in the paper may be analyzed and graduated by means of expansion in Gram's series where the generating functions may be either the Laplacean normal probability function, the Poisson probability exponential, the Kapteyn-Van Uven functions, the frequency functions of Pearson, or any other generator fulfilling certain simple *a priori* requirements. Since all the constants in the Gram series can be expressed as semi-invariants for which one can compute the standard errors according to the formulas originally derived by Thiele,* it may be of interest to contrast this latter method briefly with the one proposed by the author.

As an illustration take the observations in the author's Table 4 on the annual rainfall in Tientsin, China. These observations, thirty-two in number, may be arranged in a frequency table of the form shown in Table 27.

A calculation of the first four semi-invariants gives the following results (interval = 3 in., provisional mean at 19):

$$\begin{aligned} \lambda_1 &= 0.031 \\ \lambda_2 &= 3.147 \\ \lambda_3 &= 1.425 \\ \lambda_4 &= -6.725 \end{aligned}$$

from which,

$$\begin{aligned} \text{Mean} &= M = 18.907 \text{ in.} \\ \text{Dispersion} &= \sigma = 5.322 \text{ in.} \\ \text{Skewness} &= c_3 = -0.0425\dagger \\ \text{Excess} &= c_4 = -0.0283 \end{aligned}$$

The Gram series is, therefore, of the following form,

$$F(x)\,dx = [\phi_0(x) - 0.0425\,\phi_3(x) - 0.0283\,\phi_4(x)]\,dx$$

in which,

$$\phi_0(x) = \frac{1}{5.322\sqrt{2\pi}}\, e^{-\frac{1}{2}\left[\frac{x-18.907}{5.322}\right]^2}$$

* "Almindelig Iagttagelseslaere," Copenhagen, 1888.

† In Pearson's notation the skewness equals $-3c_3$, and excess (or kurtosis) equals $3c_4$.

and $\phi_3(x)$ and $\phi_4(x)$ are its third and fourth derivatives for which the numerical values are given in the well-known standard table by Jörgensen or in the smaller table by Glover.

TABLE 27.

Rain, in inches.	x.	Frequency.
9–12	− 3	2
12–15	− 2	5
15–18	− 1	7
18–21	0	5
21–24	+ 1	6
24–27	+ 2	4
27–30	+ 3	2
30–33	+ 4	1
Total.....	....	32

The author's formula for the "duration" curve of the rainfall is:

$$\frac{t}{100} = 1 - (10)^{-0.00193\,(x-8)^{2.10}}$$

A comparison between the observed frequencies and the respective graduations according to the two methods is given in Table 28.

TABLE 28.

Rainfall, in inches $= x$. (1)	Observed frequency. (2)	Goodrich's formula. (3)	Gram's series. (4)	Normal curve of the logarithms of observed rainfalls. (5)
Less than 9	0	0.14	0.64	0.1
9–12	2	2.37	2.43	1.8
12–15	5	4.92	4.81	5.0
15–18	7	6.28	6.43	7.0
18–21	5	6.16	6.43	6.6
21–24	6	5.00	5.26	5.0
24–27	4	3.42	3.50	3.0
27–30	2	2.00	1.76	1.7
More than 30	1	1.71	0.72	1.8

Testing for "goodness of fit" of these graduations in accordance with the Pearson criterion (which criterion—or its essentials—by the way had already been found by Helmert in the Seventies), it is found that $P = 0.9965$ in the case of the Goodrich formula, and $P = 0.9914$ for the Gram series, indicating an almost perfect and practically equally good fit for the two methods of graduation. It must be constantly borne in mind, however, that these results are only graduations pure and simple and do not give any inkling about the frequencies of future rainfalls.

As stated, the writer does not know of any method of measuring the standard errors in the Goodrich formula; but in the case of the Gram series the methods as developed by Thiele show that the standard errors for the skewness, c_3, and the excess, c_4, exceed ± 0.075 and ± 0.036, respectively.

All this goes to show that one is not justified in attaching any importance to the measures of skewness and excess (or, for that matter, to the closeness of "goodness of fit"), since the standard errors of these constants are greater than the constants themselves in the small sample of thirty-two items. Moreover, since Gram's series yields just as good a fit as the Goodrich formula, there seems good reason to demand that the author furnish standard errors for his empirically derived constants before attaching any undue importance to his results. In fact, the writer seriously questions whether the proposed method in meager samples of this kind gives more reliable results than the elementary method of finding the mean and the dispersion of the logarithms of the individual observations in conjunction with the use of the ordinary probability integral as shown in Column (5), of Table 28, which represents the formula, $\frac{32\,dz}{\sqrt{2\pi}}e^{-z^2:2}$, when $z = (\log x - 1.275) : 0.1273$.

As a second illustration take the frequency records of two series of twenty-six consecutive winnings each in a carefully performed game of chance, as given in Table 29.

TABLE 29.

Winnings, 10 *R*.	Frequency, in first series.	Frequency, in second series.
$0.40	1	0
0.50	1	0
0.60	1	0
0.70	4	1
0.80	4	8
0.90	4	5
1.00	3	2
1.10	3	2
1.20	1	4
1.30	2	1
1.40	2	2
1.50	0	1
Total.....	26	26

Through an application of the graphical method, as suggested by the author and illustrated in Fig. 35, the following results were obtained for the "duration" curve for the first series:

$$\frac{t}{100} = 1 - (10)^{-0.0000\,2235\,(10\,R - 40)^{2.854}}$$

The frequency curve proper corresponding to the duration curve is shown in graphical form in Fig. 36. No attempt has been made to fit the second series to the Goodrich formula since it is evident that quite a different "duration" curve from the one in the first series would be obtained.*

* It should be emphasized that the results of the two series as shown in Table 29 are due solely to chance and are not caused by any physical defects in the dice themselves. In fact, the two series represent the two first samples of 40 total samples, each sample containing 26 individual throws. The 40 samples when treated as a generalized Bernoullian series showed perfect agreement with the *a priori* theory. The empirical results produced a skewness which was about equal to its standard error, while the excess (kurtosis) came within 5% of that of the *a priori* theory. All this goes to show that the dice were not "loaded," and that they were thrown in a purely random fashion. The true skewness is, of course, zero, since the curve is symmetrical; the empirically derived skewness of the total 1040 throws was —0.0012.

What are the real facts in this illustration? The series previously discussed represents the monetary winnings in the following simple game of chance, namely, three dice are thrown simultaneously and the winnings equal ten times the sum of the results of the throw. In other words, one is

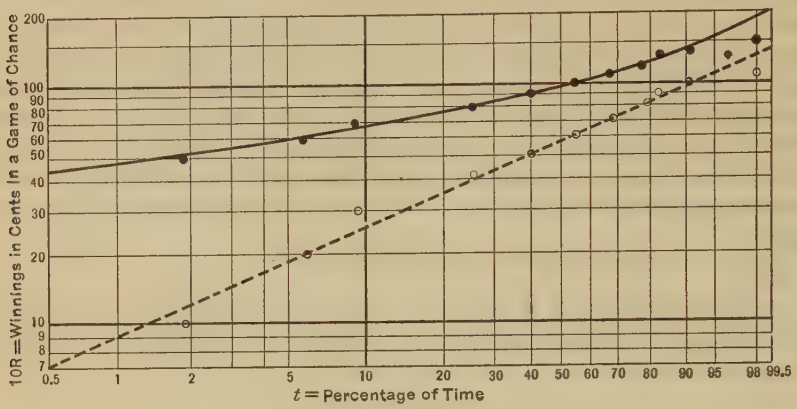

FIG. 35.—GRAPHICAL FORM OF DURATION CURVE.

faced with the classical problem of Galileo in a slightly modified form. There are altogether 216 possible cases of sums in a single throw of three dice, running from 3 to 18, the former being the event of obtaining 3 ones and the latter being the event of obtaining 3 sixes. The true *a priori* probabilities for the different winnings are as given in Table 30.

TABLE 30.

Winnings, 10 *R*.	Probability.	Winnings, 10 *R*.	Probability.
$0.30	1 : 216	$1.10	27 : 216
0.40	3 : 216	1.20	25 : 216
0.50	6 : 216	1.30	21 : 216
0.60	10 : 216	1.40	15 : 216
0.70	15 : 216	1.50	10 : 216
0.80	21 : 216	1.60	6 : 216
0.90	25 : 216	1.70	3 : 216
1.00	27 : 216	1.80	1 : 216

The probability curve (Fig. 36) can be expressed very closely by the following Gram series:

$$F(x)\,dx = [\phi_0(x) - 0.017\,\phi_4(x)]\,dx$$

in which,

$$\phi_0(x) = \frac{1}{2.958\sqrt{2\pi}}\,e^{-\frac{1}{2}\left[\frac{x-11.5}{2.958}\right]^2}$$

is the generating function and $\phi_4(x)$ its fourth derivative.*

* Since the presumptive curve is symmetrical around the mean, the skewness is equal to zero and the excess (kurtosis) as expressed by the semi-invariants of even order need only be considered.

This true presumptive curve is shown alongside the entirely differently shaped Goodrich curve in Fig. 36 and demonstrates beyond doubt that the author's statement to the effect that, "the graphical analysis of 'integral skew frequency' or 'duration curves', as herein presented, is developed especially for the examination of records consisting of from about ten to fifty observations", is a bit premature, to say the least. All that a graphical process like that of the author does in the case of small samples is to yield a curve which fits the observations but makes no adequate allowance for the errors or "deformities" which arise from chance or sampling. As shown in the illustration from a game of dice the presumptive or ideal frequency curve is of a smooth and symmetrical form in contradistinction to the author's skewly shaped curve.

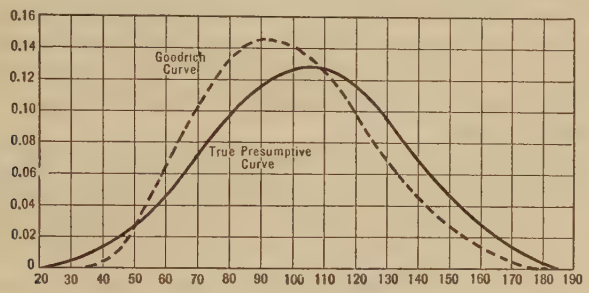

FIG. 36.—GRAPHICAL FORM OF FREQUENCY CURVE.

As the writer passes on he merely suggests that the reader consider what would happen to a gambling establishment if it were to arrange the stakes in a game of chance, such as described in the previous paragraph, on the basis of the results as derived from an application of Mr. Goodrich's method.

In a sense Mr. Goodrich is in a position analogous to that of a dressmaker trying to fit a custom or tailor-made dress to a handsome woman. The woman, however, may have some small deformities; she may be slightly bowlegged, or have a slight stoop, or a hundred of other smaller imperfections. In a sense all these imperfections are errors or deviations from the ideal form, and woe to the tailor who would have the audacity to try to fit the lady's dress so snugly as to bring out and actually accentuate such errors. Yet, that is exactly what the author has done, and the use of his graphically derived curves as exact, or even as close, representations of the true or presumptive frequency functions has no sound mathematical basis unless one is able to evaluate the standard errors in his formulas.

In the case of the Gram series it is, on the other hand, a relatively simple process to evaluate the standard errors of the semi-invariants according to the methods as devised by Thiele. These methods when applied to the series of dice throws mentioned, indicate that the standard errors, not only of the skewness and the excess, but of the mean and the dispersion as well, are subject to sampling errors to such an extent that one should be extremely cautious in using the observed results for future predictions. In this connection it might perhaps not be amiss to mention the recent researches on variations in small

samples, as carried on by the English biometricians, notably Dr. R. A. Fisher and "Student".

These researches, interesting as they are, do not in the writer's opinion approach and measure up to the more general methods by Thiele. Moreover, most of the methods of the English school are limited in the scope of their practical applications on account of the fact that they implicitly assume that the infinite universe from which the sample is drawn is normally distributed in respect to the variable attribute, an assumption which very rarely is realized, except possibly in the case of repeated precision measurements. Recently, attempts have been made to use rather refined mathematical methods in the case of as few as four or five observations. This, however, is a waste of time and is typical of the erroneous ideas among the adherents of the purely empirical school of statistics. As pointed out by Gram, when one deals with only four or five observations, it is not only easier, but also more exact, to deal directly with each individual result than to try to compress the observed facts into a mathematical formula involving the mean and the semi-invariants of higher order.

Many of the assertions by Dr. Fisher, in his exceedingly interesting and stimulating "Statistical Methods for Research Workers in Biology", do not rest on a sufficiently rigorous basis. At least in applying his own proposed method for the construction of mortality tables the writer has found that the criterions by Dr. Fisher under-estimate chance errors in small samples, and that it becomes necessary to use the more laborious, but also more powerful, criterions developed by Thiele and which he has characterized by the suggestive name of "Fejlkritik". This "Fejlkritik" is based on the hitherto little known and less appreciated theory of Thiele in connection with his formulas for the standard errors of the semi-invariants of the observations.

Allen Hazen,* M. Am. Soc. C. E. (by letter).—This paper presents a new method that is entirely graphical, for solving problems which, under the procedures previously suggested, were partly numerical and partly graphical. The procedures referred to were that of the writer† and that of H. Alden Foster, Assoc. M. Am. Soc. C. E.‡ It may be taken for granted that any of these methods applied to reliable data will produce reliable results for points within the range covered by these data. Within this range the calculations may be classed as statistics, as distinguished from estimates for values outside the data ranges where extrapolation is an essential part of the process.

It is also to be mentioned that the method used by Weston E. Fuller, M. Am. Soc. C. E.,§ was entirely graphical, but less general in character, and related only to the analysis of values near the upper end of the curve.

It will be shown how these four methods compare in computing, for example, the required spillway capacity to meet remote contingencies, when a

* Cons. Engr. (Hazen & Whipple), New York, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 214.

‡ *Loc. cit.*, Vol. LXXXVII (1924), p. 142.

§ *Loc. cit.*, Vol. LXXVII (1914), p. 564.

record of moderate length only is available as a basis of calculation. This is a case of extrapolation and is the severest test of the methods.

The basis of the calculation is to be a record of the greatest 24-hour flows of the given stream through a period of years. These greatest 24-hour flows form series with decided skews. The flood ratio for each year is the ratio of the flood for that year to the mean flood for the entire record period. It would be possible to take an actual series for this comparison; such, for instance, as the record of the Merrimac River floods,* but for this purpose take assumed values for a 25-year series, these values being as nearly as possible normal or average values for data of this kind for the Northeastern United States. The values selected in this way are as follows:

2.02, 1.67, 1.50, 1.39, 1.30, 1.23, 1.18, 1.13, 1.09, 1.04, 1.00, 0.97, 0.93, 0.90, 0.87, 0.84, 0.81, 0.77, 0.74, 0.71, 0.67, 0.64, 0.60, 0.54, 0.46.

The coefficient of variation of this series is 0.37, and the coefficient of skew computed by the method described† by the writer is 0.91. As the coefficient of skew depends on the number of terms in the series, the skew to be used in applying Foster's method is to be adjusted as described by the writer,‡ and the adjusted value to be used with the Foster factors is 1.22.

With this as a starting point, the writer has calculated from this assumed series the most probable values for points near the limits, by four methods as given in Table 31.

TABLE 31.

Percentage of time.	10	1	0.1	0.01
Computed by the writer, using the Goodrich graphical method	1.50	2.20	2.75	3.30
Foster method	1.50	2.17	2.78	3.35
Semi-logarithmic paper, Fuller method	1.50	2.25	3.01	3.77
Logarithmic probability method preferred by the writer, done either graphically or numerically	1.50	2.23	2.98	3.85

In Table 31 the results are arranged in the order of size, the Goodrich method giving the lowest values and practically checking the Foster method. The semi-logarithmic paper and the logarithmic probability paper form another pair with results not far apart and higher at the extremity.

Unfortunately, there are no data that can be used in determining which of these four methods is nearest the truth. That must be reserved for further study.

R. D. Goodrich,§ M. Am. Soc. C. E. (by letter).—In reviewing the discussions, several general considerations should be kept clearly in mind. The application of probability methods and the study of frequency distributions as applied to the solution of engineering problems are of comparatively recent origin and are not in general use among engineers.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 220.

† *Loc. cit.*, p. 215.

‡ *Loc. cit.*, Vol. LXXXVII (1924), p. 175.

§ Engr. in Chg. of Works, Chihli River Comm., Tientsin, China.

The writer became interested in the subject through the papers published by the Society, and these papers have constituted almost his only source of information. The numerous difficulties and criticisms cited in the discussions of previous papers offering methods adapted to the use of engineers, led to this attempt to develop a method, preferably graphical, which would be comparatively simple and could be easily applied for the fitting of curves to the usually short series of data available in most engineering investigations.

The study of analytical methods developed for the investigation of statistics and the fitting of curves to tabulated data, except as already noted, was purposely postponed to avoid bias as far as possible in the development of a graphical method based on empirical equations. The purpose was to find a device that would give results of sufficient accuracy for the needs of the engineer and, at the same time, would be commensurate with the quality, character, and extent of the data available. The method was developed by an engineer, it was used for a year or two, and then the paper was prepared and presented from the point of view of, and with the special needs of, the Engineering Profession always in mind. The writer hardly dared to hope that his efforts would prove of interest to such able professional statisticians as have discussed it, and their valuable contributions will be greatly appreciated by all who may study this subject, even if their points of view and the requirements for their work are so radically different from those of the engineer. The "claims" and "conclusions" referred to in more than one discussion, were put forward for the express purpose of drawing out comment, and suggesting points for independent investigations.

In considering the points raised in the several discussions, those by engineers will be dealt with first, then those of the statisticians. Finally, some additional developments connected with the process, will be given.

Mr. Streiff raises the question as to whether rainfall and run-off may properly be investigated by probability methods. While some investigators incline to the theory that these phenomena are periodic functions somewhat similar to the tides, a method of analysis on that basis has not yet been made generally available to the profession, and the theory is still questioned by most engineers as not having been proved.

The writer tried to avoid the term "probability" as far as possible, using "frequency" instead as more applicable to the work in hand. The statement that "there is an even chance that the next succeeding record will fall within these limits", referred only to the records given by the data, which are mostly annual rainfall or run-off records, or other statistical data taken from references on this general subject. The statement should be taken in connection with the context; it certainly does not refer to daily stream flow as Mr. Streiff assumes.

That "daily run-off figures are not unrelated numbers" is undoubtedly true; hence, such records are not Gaussian numbers and seldom follow very closely the normal law, based on the assumption of a very large number of independent causes producing variations in any series of observations of a

variable. As the number of causes is reduced, and they become more or less interdependent, the tendency to skewness increases. This is one of the fundamental considerations in Kapteyn's analysis of skew frequency curves, and is one basis of his criticism of the method of development of the Pearson curves.

Mr. Streiff uses the theorem of Bayes to show that a series of daily gauge heights are not "unrelated numbers", with the remark at the close of the paragraph that, "if the theorem of Bayes is true, the river flow cannot be a fortuitous sequence." While a series of daily gauge heights may not be a fortuitous sequence, this is not the place to enter into a discussion of the merits of Bayes' Rules, nor is the writer competent to do so, as it is a problem on which some of the best authorities do not agree.*

The six laws of river discharge given by Mr. Streiff, are very interesting, but until he or some other competent authority publishes a method by which definite quantitative values can be assigned to each of the elements involved, so that predictions of future maxima and minima can be made, other methods of analysis must be used. At present, the method of frequency studies seems to be gaining in favor, and it will probably continue to be used for a long time in problems of storage and investigations of flood conditions. Even if the periodic theory is shown to apply to annual or even monthly rainfall on a large area, there would seem to be a considerable element of chance as to whether very severe or local storms would strike any given smaller watershed. Additional causes might also enter to effect variations in the discharge of the smaller streams.

Although Mr. Foster's discussion is eminently fair, perhaps some of his statements would have been modified had he a more complete knowledge of the use, scope, and flexibility of these methods than could be obtained from such a brief paper and from the study of the few curves shown on the diagrams. First, the method of selecting the constants might be more explicitly stated somewhat as follows: In any actual case, an approximate selection of one or two of the constants is made, and a straight line is located, by eye, or by the method of centroids, so as to pass through these points as closely as possible; the two remaining constants are then determined graphically.

The equations developed by Professor Pearson and the methods of fitting the curves of these equations to actual data by means of moments, will always have their proponents, but any one who is about to use these methods on account of their "strong mathematical basis" should not be misled by claims of great advantage. Mr. Foster quotes from Elderton who is one of the foremost proponents of the Pearson system. There are, however, other viewpoints; for example, Mr. Fisher states† that,

"The immense production of Pearson coupled with his well-nigh perfect and forceful style of writing has to a certain extent overshadowed the researches of his compatriot, Edgeworth, whose works, according to the Danish actuary and mathematician, Jörgensen, are greatly superior to those of Pearson both in scientific rigor and practical application."

* Any who are interested in this question may refer to Chapter VI of the excellent work on "The Mathematical Theory of Probabilities," by Arne Fisher.

† "The Mathematical Theory of Probabilities," by Arne Fisher, p. 185.

The writer has referred to the papers on skew frequency curves by the Dutch astronomer, Kapteyn, who states in the "Introduction" to his First Paper:

"Pearson's curves can only be considered as purely empirical representations". * * * From several passages in the paper quoted (Vol. 186, *Philosophical Transactions*), it seems that Pearson himself is inclined to admit this empirical character of his curves."

Still another quotation may perhaps be pertinent, as Mr. Foster insists so strongly on the "sound mathematical basis" of the Pearson equations. The following is from the recent book by Dr. R. A. Fisher, Chief Statistician of the Rothamsted Experiment Station.* In discussing the fitting of curves to normal distributions, Dr. Fisher states:

"Fitting by moments has also been widely applied to skew curves, and others which are not normal; but such curves have not the peculiar properties which make the first two moments especially appropriate, and where the curves differ widely from the normal form, the above two statistics may be of little or no use."

The two "statistics" referred to are the mean and standard deviation.

Mr. Foster calls attention again to two reasons why the Pearson equations are not well adapted to use in engineering work. The first of these is entirely obviated by the equations now proposed, as the equation of the general differential curve can be integrated for all real values of the constants; and, incidentally, this one equation covers the entire practical range of conditions instead of the four in the Pearson system of skew curve equations. Secondly, in using the tables by which Mr. Foster has made it possible to plot duration curves for the two most common types of Pearsonian skew curves, certain limitations should not be entirely forgotten. While those for the Type III curve are theoretically correct for the particular correlation between the coefficients of skew and variation on which the method is based, this condition must first be established; while in the writer's method it is unnecessary to determine any relation between these coefficients.

The correlation established for these coefficients as used in developing the theoretical relations for the utilization of the Type I curves is made on the assumption that average conditions obtain and that actual conditions do not depart from this assumption enough to cause any appreciable error. In addition, it must always be remembered that, as Mr. Foster states, "the computed values of the constants are subject to a certain amount of error", which may or may not be large. This will depend mainly on the regularity of the data, and while this error exists in all data, by whatever method examined, the selection of a suitable curve by the writer's method does not depend on the use of any computed criteria or the assumption that the data have positive skewness. Probably all engineers will agree that the best test of any method is whether the curves obtained fit the data, and the "relative simplicity and convenience" of the method used.

* "Statistical Methods for Research Workers in Biology," by Dr. R. A. Fisher, 1925, Oliver & Boyd, Edinburgh and London.

A little practice in plotting on skew frequency paper and the comparison of a few of the diagrams in the paper by Mr. Foster* and in that of the writer, will show great similarity in the shape of the final curves. The data for West Canada Creek plot in the same sort of reverse curve, while the curves for the Murray River are also similar. It is only necessary to allow for the fact that the run-off scale is arithmetic in one case and logarithmic in the other. For examples fitted by the Pearson Type III equation, in addition to the shape of the curve as disclosed by the original data as a general guide to the skewness, the slope of the auxiliary curve furnishes a complete measure of this characteristic. As stated in the paper, when the slope with respect to the R-axis is about 3.3121, the curve will be practically normal. In fact, for values of the index, c, from about 2.5 to 4.5, data could often be plotted on normal probability paper and a straight line drawn without the use of Mr. Foster's tables, since, for small samples, the probable error of the coefficient of skew might easily be greater than its computed value. If the slope of the auxiliary curve on skew frequency paper is less than 3.3, the skewness is positive, while for values greater than 3.4, it is negative.

It seems to the writer that the "spread" of the data is as clearly indicated by the range of the complete data when plotted on skew frequency paper as on normal paper, and that the percentage of time between the mean and the median is also as easily and clearly obtained in one case as in the other, if these measures of the variation and skewness are desired for general information as to the character of the curve. Neither method seems to have any advantage on this point, and the selection should rather be made as to ease and convenience in use and the results obtained.

Mr. Nimmo's discussion offers an opportunity to contrast the use of the graphical aids suggested in the paper for the determination of the mean, with the method which Mr. Shewhart seems to think is recommended. With nine years' records, Mr. Nimmo constructs curves that are similar to those of Fig. 3, and by the use of Equation (5) he arrives at a value for the mean about 1.3% greater than his former estimate, which was based on entirely different considerations and was about 3.5% greater than the mean of the tabulated data. The fact that the original independent estimate is also greater, tends to show that the true mean is probably greater than that of the sample, as indicated by the parameters of the curve constructed by the writer's method.

The writer believes that the duration curve shown in Fig. 23 points out the probable frequencies for high and low values of annual run-off to be expected for a range of from about 2% to 98% of the time, quite as well as they could be obtained by any other method, and within the limits of error for a series of this length. The approximate frequency curve calls attention to the much greater probability of years with low, than with high, run-off. For such a short series as is here shown, the curves are of little practical value outside the range suggested, unless attending circumstances, such as Mr. Nimmo mentions, give the series very unusual range and strength.

* "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

While the effect of "fluctuations in sampling" will undoubtedly cause considerable differences in the frequency distributions for the shorter series in stream flow data as Mr. Hall points out, the marked differences, which vary from the practically normal curves for some streams in the Eastern States to the very extreme skewness of some of the Californian and Australian rivers, cannot be accounted for in this way. The writer believes that the combining of rainfall or run-off records from different localities, sometimes widely separated, as has been done in certain cases to build up a long series, is permissible only when similarity can be fully demonstrated. While comparisons are undoubtedly necessary and valuable, most engineering works connected with hydraulic development will depend on the results of an intensive study of conditions for the particular drainage area involved.

If the primary purpose of an investigation is that of correlation, the coefficients and analytical methods already developed may prove most useful; and graphical methods, as far as they go, may be used for obtaining curves for comparison and study. Previously developed graphical methods are not broad enough for the correlation of all the observed conditions of stream flow. Mr. Hall himself furnished data that were used to illustrate three cases (Figs. 4, 5, and 9) of stream-flow distribution, which the previously devised graphical methods do not cover. If the purpose is to arrive at an understanding of the causes operating to produce the fluctuations in the observed results, then it is quite possible that new methods may prove useful. This was suggested in the paper but not demonstrated fully because of insufficient opportunity. One instance of this kind in the writer's experience (the Yung Ting Ho) was mentioned in the discussion by Mr. Goodrich, and it is hoped that others may make a similar use of the method.

It was stated in the paper that the two lowest records were omitted from the data in Table 8 for the run-off of the Murray River. This was done to illustrate a duration curve for which the frequency curve was the special one-sided type shown in Fig. 6. The duration curve corresponds to the one in Fig. 8* of Mr. Foster's paper for which the $c\,s$ is 2.0. If the writer's curve had been drawn with a lower limit of less than 20% of the mean so as to include the lowest records, the skewness would have been less, so that the frequency curve would have resembled that in Fig. 3. In Fig. 6 the data depart from the curve for percentages of time between 60 and 80, as well as at the lower end of the curve. These departures cannot be described by a single curve, but both could probably be covered by the use of combination curves, as was done in the Yung Ting Ho investigation already cited. This points to the probability that at least two distinct sets of causes operate to produce the fluctuations observed in the discharge of the Murray River.

Mr. Hall gives a most useful table (Table 21) with a formula derived to determine the probable error in the percentage of time corresponding to any given value of the variable for which an integral frequency curve has been drawn. The formula for the mean error of the proportion, p, of the total frequency of a series of events, is also given with its derivation by Fisher,†

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 169.

† "The Mathematical Theory of Probabilities," by Arne Fisher, p. 108.

as $E = \sqrt{\frac{p\,q}{s}}$, in which, p is the given relative or proportional frequency, q is $1 - p$, and s is the number of items in the series.

This value, s, should be clearly distinguished from the n in the writer's equations. In the latter, n is more accurately described as the "total population of the given universe", which may be 100% or unity, and is the maximum ordinate to the duration curve. To reduce the mean error, E, of the relative frequency, p, to the mean error, E_t, of the percentage of time, t, it is only necessary to multiply by n, or 100%, as was done by Mr. Hall. The formula then becomes $E_t = n\sqrt{\frac{p\,q}{s}}$, which is, as he states, a general formula applying to all curves, however derived. Hence, values of the probable error, which includes the factor, 0.675, on the right of the equality, may be laid off from points on a curve, in a direction parallel to the t-axis, as in Fig. 37.

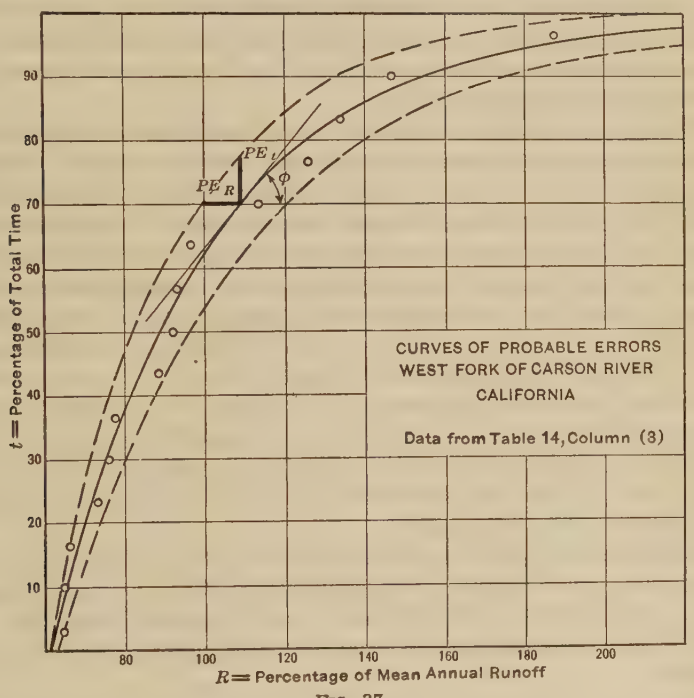

FIG. 37.

The data for Fig. 37 are from Column (3) of Table 14, and the curve shown by the solid line was plotted from an auxiliary curve located on skew frequency paper by the centroid method. Values of the probable error, PE_t, were computed from the formula for E_t, for a series of values of p, with s equal to 15. With these values of PE_t, the two outer curves were plotted. It is evident that these two "probable error curves" must also describe the probable error, PE_R, of the value of the variable, R, corresponding to any given value of t, as given by the duration curve, with at least a fair approximation.

A formula for the probable error of the variable in terms of p, q, n, and s, is easily derived and can be used when a total frequency or duration curve has been drawn by any method whatever. From Fig. 37 it is clear that if ϕ is the angle of inclination of the tangent to the duration curve with respect to the R-axis, for any corresponding values of the variables, then:

$$\tan \phi = \frac{d\,t}{d\,R} = t' = \frac{\Delta\,t}{\Delta\,R} = \frac{P\,E_t}{P\,E_R}$$

Hence,

$$P\,E_R = \frac{P\,E_t}{t'} = \frac{n}{t'}\sqrt{\frac{p\,q}{s}}$$

in which, t' is the ordinate to the frequency or differential curve for the given value of R; or it may be taken approximately from the slope of the duration curve. This formula can easily be reduced to the general formula given by Yule,* which fact should give it sufficient corroboration for practical use.

With these formulas one can test the probable error, $P\,E_R$, of the variable, R, as well as the percentage of time, t, for any point on a duration curve; or, error curves may be drawn as in Fig. 37, giving a clearer conception of the reliability of the graphical results.

The formulas give the probable error of a single record, or of any point on the central curve, and show that this error varies constantly from point to point along the curve, while PE is a constant for any value of the percentage of time for all curves, whatever their skewness. For any given value of t (or p) $P\,E_R$ for different curves varies inversely as the slope of the curve with respect to the R-axis. Thus, where the slope is steep, showing an accumulation of points for a small change in the value of R, as at the left of the diagram, $P\,E_R$ is relatively small, while at the right-hand end of the curve, where the points are scattering and the curve is flat, $P\,E_R$ is very large. On the other hand, a considerable change in the value of R at this end of the curve has very little effect on the value of, and probable error in, t. Hence, whether the upper limit of this curve was taken as 400% of the mean annual run-off, or even as 500%, the difference in the duration curve for 98% of the time would be comparatively small.

If "probable error curves" be plotted as shown in Fig. 37 and all the points used in deriving the duration curve fall between the error curves, the data would seem to be fairly represented by the curve obtained, although, as Mr. Hall points out, deviations up to three times the probable error may sometimes occur.

While the curve representing the true distribution for the annual run-off of the river may lie partly or even wholly outside the limits defined by the two "probable error curves", there is an even chance that it lies between them. "Limiting error curves" may also be drawn at distances of two or three times the distance, PE_t, from the duration curve, with increased probability that the true curve lies between them. The examples of the use of the formula, as given by Mr. Hall, will help to make clear the use of these "error curves".

For the small samples with which engineers are usually compelled to deal, it is better to substitute $(s-1)$ for s in these formulas. This would bring

* "Introduction to the Theory of Statistics," p. 337.

them into line with the generally accepted formulas of least squares, the forms already given being reserved for use with large samples only. For ordinary use, the formulas would then be written:

$$P\,E_t = n\sqrt{\frac{p\,q}{s-1}};\quad P\,E_R = \frac{n}{t'}\sqrt{\frac{p\,q}{s-1}} \qquad (11)$$

Mr. Hall's formulas and suggestions as to the determination and use of the probable error in the percentage of time with the added formula for the probable error of the corresponding value of the variable, should be sufficient for most practical applications of any graphical method.

It does not seem to be possible to derive any formula for the probable error of the index, c, itself; but by making the assumption that a certain method is followed for drawing the auxiliary curve after the observations have been replotted, using the most suitable values of the limits, it is possible to develop what seems to be a logical expression for the probable error of the reciprocal of the index, and from this the desired information can be obtained as to the reliability of the value of this parameter.

The method of centroids was suggested in the paper, and if it is used the value of c can be computed from the data as follows:

Assume that the S-observations are divided into two groups. In the first group of r-observations, none is greater than R_r, while the second group contains those greater than R_r. Then, if $\log f\,(R)_1$ and $\log y_1$ are the averages of logarithms of $f\,(R)$ and y in the first group, and $\log f\,(R)_2$ and $\log y_2$ are the averages for the second group, the corresponding values of $f\,(R)$ and t will be the co-ordinates of the centroids of the groups of points as they are plotted on skew frequency paper. Then, the value of c can be computed from the relation:

$$\frac{1}{c} = \frac{\log f\,(R)_2 - \log f\,(R)_1}{\log y_2 - \log y_1}$$

This formula applies when the percentages assigned to each observation are determined for small samples by the method of the paper, as described in connection with Fig. 1 and Table 3. It is evident that the value of the denominator of the right-hand member is a constant for all samples containing the same number of items or observations, irrespective of the values or form of $f\,(R)$. Hence, the mean error E_{rc}, of the reciprocal of the index, c, will be this constant times the square root of the sum of the squares of the mean errors of the two terms in the numerator of the right-hand member. The true values of the means, $\log f\,(R)_1$ and $\log f\,(R)_2$, would be obtained if the true values of R were taken from the true curve and the means of their logarithms computed. Then, for each group, the mean-square error could be obtained by taking the mean of the squares of the differences of the logarithms of the observed and true values of the variable for the given values of t, in each group. If v represents any difference,

$$E_{rc} = \frac{1}{\log y_2 - \log y_1}\sqrt{\frac{\sum_{s-r}^{s} v^2}{s-r} + \frac{\sum_{1}^{r} v^2}{r}}$$

in which, r is the number of items in the lower group, and $s - r$, the number in the upper group.

If the two groups are made equal, as they can be with very good results, this formula can be simplified to read,

$$E_{rc} = \frac{1}{\log y_2 - \log y_1} \sqrt{\frac{\Sigma v^2}{\frac{1}{2} s}}$$

As the auxiliary curve usually will be only an approximation to the true curve, $\frac{1}{2} s$ may be replaced by $\frac{1}{2} s - 1$ to conform with the adjustment made in the theory of least squares in computing the standard deviation from a limited number of observations, and the formula becomes:

$$E_{rc} = \log y_2 - \log y_1 \sqrt{\frac{2 \Sigma v^2}{(s - 2)}}$$

Since the probable error is 0.675 times the mean error, the coefficient becomes $\frac{0.675}{(\log y_2 - \log y_1)}$.

If $\frac{1}{c} = k$ and $\pm E$ is the probable error of k, then there will be an even chance that the true value of c lies between $c = \frac{1}{k + E}$ and $c = \frac{1}{k - E}$. The slope of the auxiliary curve, as determined by this method, will not be exactly midway between these two values of c, but will be nearer one than the other, depending on the size of the angle between the R-axis and the auxiliary line. If the probable error sought had been the error in this angle, the application of equal positive and negative corrections to the angle would produce unequal positive and negative changes in the value of the tangent of the angle. The formula as derived produces this result, which tends to strengthen one's confidence in it.

From the formula for the probable error of t, evidently this error becomes zero at the limits, while that for R becomes indeterminate. The standard errors of the limits themselves may be indeterminate also; at any rate it does not seem to be possible to derive any formulas applicable to them. However, a little consideration of the graphical meaning of the formula for the probable error of the reciprocal of the index, shows that this error will be a minimum for any given series of observations when the best values of the limits have been selected. If the best values have not been selected, the points will depart farther from a straight line when they are replotted to draw the auxiliary curve, and Σv^2 will then become greater. Hence, the most probable values of the limits would seem to be those which will make $P E_{rc}$ a minimum. The reliability of the values selected or determined for the limits is, therefore, indirectly determined by testing the value of c. The probable error of c, can also be determined approximately by the foregoing formula when the auxiliary line has been located by eye instead of by actually finding the co-ordinates of the two centroids.

Applying this formula to the data of Table 4, which was used for comparison by Mr. Fisher, it is found that $\frac{1}{c}$ is 0.4762 with a probable error of $\pm$ 0.0089, so that there is an even chance that the value of c is between 2.06 and 2.14.

The following values of $\frac{0.6745\sqrt{2}}{(\log y_2 - \log y_1)}$ may be useful in the application of the formula for the probable error of $\frac{1}{c}$.

Values of:

$$S = 10 = 20 = 30 = 50$$

Values of:

$$\frac{0.6745\sqrt{2}}{\log y_2 - \log y_1} = 1.184 = 1.158 = 1.151 = 1.150$$

For other values of S, the required coefficient can be found by extrapolation, but great accuracy is not warranted with small samples, and a sufficient degree of approximation would probably be obtained by adopting the average value of 1.16.

For very large samples the general formula should be used and the means of logarithms for the values of $f(R)$ and $1 - \frac{t}{100}$ can be computed with weight assigned to each point in proportion to the frequency in each class, if extreme accuracy is required. Otherwise, a simple grouping can be made with about one-half the total frequencies in each group.

For graphical work, the differences, v, can be most quickly and easily taken directly from the diagram for the auxiliary curve by preparing a scale of 100 "equal parts" corresponding to the length of one complete cycle of the logarithmic scale for $f(R)$. In this way all computations are reduced to a minimum and can be handled with an ordinary slide-rule.

For the mean error of h, or more exactly of $\log h$ write the equation in the log colog form:

$$\log \operatorname{colog}\left(1 - \frac{t}{n}\right) = \log h + c \log f(R)$$

Log h and c become the parameters and define the curve just as well as h and c.

The mean error, E_c, can be obtained from the value of E_{rc} as already given:

$$E_{rc} = \frac{\sqrt{\frac{2\Sigma v^2}{(s-2)}}}{\log y_2 - \log y_1}$$

With y equal to $\operatorname{colog}\left(1 - \frac{t}{n}\right)$, $\log f(R)_m$ is the average of logs of $f(R)$. Then:

$$-\log h = c \log f(R)_m - \log y$$

The value, $-\log y_1$ will be a positive quantity since y will always be less than unity, and the quantities in the equation will generally have the geometrical relations shown on Fig. 38 since c is the tangent of the angle, ϕ; but $-\log y$ is a constant depending only on the number of items in the series for small samples handled as given in the paper, so that the location of the vertical through the center of gravity of all the points, as plotted on skew frequency paper, is fixed by the number of points. By the method of drawing the auxiliary line used in deducing the formula for the mean error of $\frac{1}{c}$, this line will pass through the C. G. of all the points (Fig. 38). The mean error of a variable plus a constant is equal to the mean error of the variable itself. Hence:

$$E_{-\log h} = E_{\text{colog } f(R)_m}$$

The square of the mean error of the product of two variables, however, is the sum of the product of each variable by the mean error of the other. Hence:

$$(E_{-\log h})^2 = [\log f(R)]^2 (E_c)^2 + \frac{c^2 \Sigma v^2}{(s-1)}$$

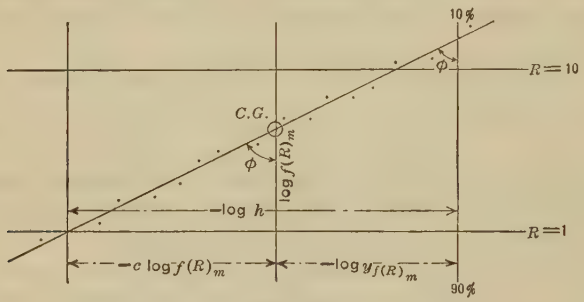

FIG. 38.

Having found the mean or probable error of $\log h$, one can compute the values of h on each side of the given mean value and find the mean error of this quantity. The mean error of $\log h$ will increase as $\log f(R)_m$ increases, with the mean error of $\log f(R)_m \times c$ added.

Applying these corrected formulas for the probable errors of these two parameters to the equation for the annual rainfall at Tientsin one gets $c = 2.10 \pm 0.12$, approximately, and $h = 0.00193 \pm 0.00064$. With these values for these errors, there is much reason for confidence in the equation and curve since a small change in the lower limit will not have a very large effect on these errors.

As these parameters have mean and probable errors of lower order than the parameters for the skewness and excess, it would seem to put the older analytical methods based on the extension and generalization of the normal curve at a decided disadvantage when skew frequencies are being investigated.

These formulas should also, in part at least, meet the insistent demand of statisticians for an evaluation of the standard errors of the writer's formulas.

All investigators will agree that extrapolation beyond the range of actual data is the most difficult (or dangerous) of the problems connected with the study of observations of almost any kind. Yet this is the type of problem most frequently met when the observations relate to engineering problems. Mr. Hazen's comparison of the results by the writer's graphical extrapolation process, with those of the three methods previously proposed, therefore constitutes a severe and interesting test. The closeness of the check with Mr. Foster's method is gratifying to say the least, as the soundest mathematical foundation is claimed by him for that method. For frequencies of 1 in 100, 1 in 1 000, and 1 in 10 000, the largest and smallest values in each column of Table 31 differ by only about 3%, 8%, and 16%, respectively. Considering the size of the sample and the extent to which the curves were extended in order to develop these differences, they are so small that the choice of method becomes a matter of individual preference based on ease of application or familiarity with a given procedure. For one who is just beginning the application of frequency methods to engineering problems, a study and comparison of all these methods and the selection of the one that most nearly meets his individual needs, would seem to be the most logical course.

The writer is much indebted to Mr. Goodrich for his great assistance. His very careful and complete résumé of the subject requires but little comment. This is in a form that should be especially helpful to the novice interested in the graphical study of frequency distributions.

As Mr. Goodrich states, the primary reason for suggesting and publishing these particular formulas was the rather surprising fact that wherever comparative tests could be made with published results of curve fitting, the writer's curves fitted the data just as well if not better than others. Furthermore, every engineer is trained in graphical methods of investigation and these methods would require no tools or tables with which he is not already familiar and which he does not already possess. Most of the calculations required in ordinary practice can be made with four-place logarithms and a 10-in. slide-rule. He is fully equipped if he adds to these a table of the logarithms of the gamma function which is available in the publications of the Society.*

Fig. 26 and Table 22 show clearly why certain data may plot as closely approximating a straight line on skew frequency paper and at the same time be either actually or apparently subject to variation in accordance with the normal law. This is explained by the fact that each formula for these skew curves covers the case of either positive or negative skewness (mode greater, or less than, mean), depending on the values of the parameters. As the transition is made from skewness of one sign to that of the opposite sign by varying the value of the index, c, the special case occurs of an unsymmetrical curve for which the mode is equal to the mean, giving zero skewness (in the normal curve the median is also equal to the mean, but not here, although the difference may be very small). In the case of Equations (S) and (A), the departure from symmetry is very small as shown by Fig. 26 and Table 22 for certain values of h. On the other hand with Equation (B), the lack of symmetry may be very marked at the point of transition.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), Table 1, pp. 152–153.

Mr. Goodrich points out a fact often lost sight of, that, with certain physical observations, positive and negative errors do not always occur in equal numbers, although the normal law is almost universally used for the investigation and adjustment of such errors. Mr. Fisher goes still further and states that a normal distribution "in respect to the variable attribute [is] an assumption which very rarely is realized, except possibly in the case of repeated precision measurements." Hence arises the particular usefulness of methods of investigation by means of a system of unsymmetrical or skew curves.

The resolution into two frequency curves to account for the variation in gauge heights on the Yung Ting Ho was of such interest that an attempt was afterward made to correlate these two curves still more closely with the two principal storm sources, by a determination of the percentage of the rainy days of the freshet season due to each class of storms. Every effort was made to secure an unbiased estimate and the proportions were not computed until all the available records had been examined and classified. The investigation gave 61½% of the total number of rainy days as due to storms of Pacific origin and 38½% as of interior origin. While no importance is attached to the exact percentage, the variation in gauge heights appears to be accounted for with considerable accuracy for six consecutive summers by combining two frequency distributions in about these same proportions. It was also found that the severest storm on this water-shed during that time was of interior origin, corresponding with the higher range of the 40% frequency curve.

Adequate reply to the discussion by the trained statisticians who have shown an interest in the paper, is made more difficult by the difference in the points of view, and in the problems and the requirements of the two professions. The writer takes exception to the claim of Mr. Shewhart that he has proved the breakdown of the method. If a sufficient number of tests had been made on appropriate data (for the examination of which the writer's method was intended), and if the curves and conclusions based on this method had been compared with curves and conclusions based on exactly the same data and conditions and derived by other methods; and if, from this comparison, the present method had been shown to give less reliable curves and had pointed to erroneous conclusions more often than the comparative method, then the writer would be only too glad to admit that his method was of questionable value. Certainly Mr. Shewhart could have had no desire to mislead the Engineering Profession, or any other profession, by the exploitation of useless methods of investigation, or by condemning graphical analysis. On the other hand there seem to the writer to be very good grounds to believe that the graphical method is equal to any single analytical method for fitting curves to such data as are distributed according to unsymmetrical or skew frequency laws.

When Mr. Hazen gave the technical professions the benefit of his labor and genius by publishing his normal probability paper,* he did not claim that one could detect the presence of a small sample drawn from a rectangular universe

* "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

by plotting on that paper; nor has his valuable contribution to engineering knowledge ever been criticized on that account as far as the writer is aware. Nor has any such extravagant claim been made in the paper as to the use of skew frequency paper. The writer is quite ready to admit that when any such inappropriate data as Samples 1, 2, and 3 of Table 19 are plotted on skew frequency paper, they will not give "very reliable information" any more than when the same data from a rectangular distribution are plotted on normal paper.

The samples drawn from the normal universe, however, present a different case. Samples 5 and 6 plot very nearly on a straight line, both on skew frequency paper and on normal probability paper, and with either sample quite "reliable information" is obtainable as to the probable character of the universe, by either method of plotting. Fig. 26 and Table 22 show why even the method developed for the examination of skew distributions would point to the probable presence of test samples from a normal universe.

It is easy to show that Sample 4 is a very unusual one, which is only likely to occur in dealing with hundreds of small samples. By applying the elementary principles of probability found in any college algebra it becomes apparent that, if the sample represented 10 consecutive years of total annual rainfall (re-arranged in order of magnitude as shown) such a sequence of 10 years in which the amounts are all greater than the true mean, is not likely to occur more than once in 1 000 years.* With the very common conditions as to rainfall and run-off, in which years of less than the average amount are more frequent than years above the average, the probability of such a series as this Sample 4 becomes still more remote. Hydraulic engineers need have little cause for anxiety that their short records are as unreliable as Mr. Shewhart tries to show.

The writer went to great pains in developing formulas for the mean, the standard variation, etc., with suggestions for the graphical determination of these parameters when the formulas do not apply. The fact that Mr. Nimmo makes use of the formula for the mean indicates that there is no uncertainty as to the purpose of including this matter. Yet Mr. Shewhart would seem to imply that the writer recommends the use of certain quantities which "may be used for classifying curves as well as for obtaining general information about an individual curve", as substitutes for the usual parameters in one form of the equation of the normal curve. The writer does not even suggest any such use of the median and interquartile range. Hence, while Mr. Shewhart's discussion of the probable error of these quantities may have some value in showing the absurdity of using them as he tries to do, on the other hand, if this is intended as further proof of the "breakdown" of the graphical method, the absurdity of this criticism is likewise great.

As to Fig. 22 which Mr. Shewhart submits for his final proof of the failure of the method, the writer is completely at a loss to follow his argument or to

* If $p = \frac{1}{2}$ is the proportion of the universe which is greater than the mean (which is true for any normal universe), the probability that ten consecutive items drawn from the universe will all be greater than the mean, is $p^{10} : 1 = 1 : 1\,024$.

understand his attitude or purpose. From a careful study of Mr. Shewhart's paper* in connection with his comments on Fig. 22, it appears that he demands much more of the graphical method of fitting curves to data than he does of any analytical method of accomplishing this end; and he appears to have read into the writer's paper certain ideas which the writer had no thought of conveying.

On the other hand, to substantiate the writer's Conclusion 2 (page 43) a better example could hardly be found than that contained in the data used by Mr. Shewhart in preparing Fig. 22. These data are reproduced in Columns (1) to (17) of Table 32, in which, on the right, for comparison, are given the writer's calculations (Columns (18) to (26)) based on the equation derived by his graphical method. The starred values of t in Column (21) were used in locating the auxiliary curve for determining the parameters.

While Mr. Shewhart does not show the detailed calculation of the theoretical frequencies in Columns (10) and (14), all the writer's calculations are given in Table 32 just as they appeared on the single working sheet used. The computations were made with a Thacher calculator and four-place logarithms, giving an accuracy ample for the purpose, as well as being all that is warranted by the quality of the data. Mr. Shewhart's computations are carried out to six places of decimals, while the moments are computed and tabulated to the last figure. This may be necessary to secure sufficiently accurate results when fitting curves by the Gram-Charlier or Pearson methods, but it gives an appearance of reliability which may be very misleading to an inexperienced reader.

A comparison of the average deviations, as given by pointing off the last figure at the foot of Columns (11), (15), and (25), of Table 32, and the values of χ^2, which are given at the bottom of Columns (13), (17), and (26), show conclusively that the equation obtained by the simple graphical method fits this distribution more closely than either of those obtained by the most refined of analytical methods. The graphical determination leads to exactly the same conclusion, when Professor Pearson's tests for "goodness of fit" is applied to the results, as is reached with the more elaborate analytical methods namely, that the equations derived do not define frequency distribution laws such that any one of them would constitute a sufficient hypothesis to account for the variation in the observed values of the variable. This leads Mr. Shewhart to suspect that the "control" of the product has not been uniform, or in mathematical terms, that the sample probably was not drawn from homogeneous material or was not taken under uniform conditions. Dr. R. A. Fisher points out† that the result of a test of goodness of fit, such as is obtained with this sample, does not prove anything except the "insufficiency of the hypothesis". In the case here cited, Mr. Shewhart deduces the absence of "uniform control", or the non-homogeneous character of the product, by examination and comparison of the twelve monthly sets of tests which are combined to give the frequency distribution shown in Table 32.

* "Quality Control Charts," by W. A. Shewhart, *Bell Technical Journal*, October, 1926.
† "Statistical Methods for Research Workers in Biology," 1925.

TABLE 32.—Comparison of Computations by Analytical and Graphical Methods.

(1) Quality (Ft) $\frac{x}{\Delta x}$	(2) CELL BOUNDARIES LOWER	(3) CELL BOUNDARIES UPPER	(4) DEV. IN CELLS FROM ORIGIN $\frac{x}{\Delta x}$	(5) OBS. FREQ. y	(6) yX	(7) yX^2	(8) yX^3	(9) yX^4
-6.5	-5.75	-5.26	0	13	0	0	0	0
-5.0	-5.25	-4.76	1	10	10	10	10	10
-4.5	-4.25	-4.26	2	8 74	16	32	64	128
-4.0	-4.25	-3.76	3	43	129	387	1161	3483
-3.5	-3.75	-3.26	4	100	400	1600	6400	25600
-3.0	-3.25	-2.76	5	815	4075	20375	101875	509375
-2.5	-2.75	-2.26	6	1761	10566	63396	380376	2282256
-2.0	-2.25	-1.76	7	2397	16779	117453	822171	5755197
-1.5	-1.75	-1.26	8	3431	27448	219584	1756672	14053376
-1.0	-1.25	-0.76	9	3703	33327	299943	2699487	24295383
-0.5	-0.25	-0.26	10	2165	21650	216500	2165000	21650000
0	-0.25	0.24	11	510	5610	61710	678810	7466910
0.5	0.25	0.74	12	77	924	11088	133056	1596672
1.0	0.75	1.24	13	15 94	195	2535	32955	428415
1.5	1.25	1.74	14	2	28	392	5488	76832
Σ				15050	121157	1015005	8783525	78143637

(10) From Charlier (11)	(12) (2 terms)	(13)	(14) From Pearson Type IV	(15)	(16)	(17)
THEOR. FREQ. y_t	$y - y_t$	$(y-y_t)^2$	$\frac{(y-y_t)^2}{y_t}$	THEOR. FREQ. y_t	$y - y_t$	$(y-y_t)^2$	$\frac{(y-y_t)^2}{y_t}$
			/				
86	-12	144	1.674	103	-29	841	8.165
255	-155	23409	92.526	229	-129	16641	72.648
675	140	19600	29.037	631	184	33856	53.655
1466	295	87015	59.362	1463	298	88804	60.700
2668	-265	70225	26.321	2708	-311	96721	35.717
3679	-248	61504	16.718	3716	-285	81225	21.858
3441	262	68644	19.949	3477	226	51076	14.690
2024	141	19881	9.823	1997	168	28224	14.133
683	-173	29929	43.830	626	-116	13456	21.495
81	13	169	2.086	100	-6	36	0.360
			301.376	15050	1752		303.441
NO. L. CELLS = 10		P = 0.00000		NO. OF CELLS = 10		P = 0.00000	

(18) R	(19) y	(20) Goodrich (21) Graphical Determination: y %	t %	(22) Computed t %	(23) From y %	(24) equation y_2	(25) of curve $y - y_2$	(26) $\frac{(y-y_2)^2}{y_2}$
0	13							
1	10							
2	8							
3	43 74	0.49	0.49	0.38	0.38	57	+ .17	.507
4	100	0.66	1.15	1.88	1.50	226	-126	70.26
5	815	5.41	6.56	6.29	6.41	664	+151	34.34
6	1761	11.70	18.26	16.33	10.06	1510	+251	41.73
7	2397	15.93	● 34.19	34.19	17.86	2688	-291	31.50
8	3431	22.80	56.99	58.38	24.19	3641	-210	12.11
9	3707	24.63	81.62	81.40	23.02	3465	+242	16.90
10	2165	14.38	96.00	95.10	13.70	2062	+103	5.15
11	510	3.39	● 99.39	99.40	4.30	667	- 77	9.17
12	77							
13	15 94	0.61			0.60	90	+ 4	0.18
14	2		100.00	100.00				
	15050	100.00			100.00	15050	1872	22641

MOMENTS ABOUT ORIGIN O

$\mu_1' = \frac{\Sigma yX}{\Sigma y} = \frac{121157}{15050} = 8.050299$

$\mu_2' = \frac{\Sigma yX^2}{\Sigma y} = \frac{1015005}{15050} = 67.442193$

$\mu_3' = \frac{\Sigma yX^3}{\Sigma y} = \frac{8783525}{15050} = 583.622926$

$\mu_4' = \frac{\Sigma yX^4}{\Sigma y} = \frac{78143637}{15050} = 5192.368759$

NOTE: THE ORIGIN B IS THE MID-VALUE BETWEEN THE BOUNDARIES OF THE CELL CHOSEN

UNCORRECTED MOMENTS ABOUT ARITH. MEAN

$\mu_1 = 0$

$\mu_2 = \mu_2' - \mu_1^2 = 67.442193 - 64.807214 = 2.634979$

$\mu_3 = \mu_3' - 3\mu_1'\mu_2' + 2\mu_1'^3 = 583.622926 - 1628.789457 + 1043.436510 = -1.730023$

$\mu_4 = \mu_4' - 4\mu_1'\mu_3' + 6\mu_1'^2\mu_2' - 3\mu_1'^4 = 5192.368759 - 18792.356164 + 26228.484274 - 12599.963284 = 23.432505$

CORRECTED MOMENTS ABOUT ARITH. MEAN (SHEPPARD'S CORRECTIONS)

$\mu_2(\text{corr.}) = \mu_2 - \frac{h^2}{12} = 2.634979 - 0.083333 = 2.551646$

$\mu_4(\text{corr.}) = \mu_4 - \frac{1}{2}h^2\mu_2' + \frac{7}{240}h^4 = 23.432505 - 1.317490 + 0.029167 = 22.144232$

$h = 1$

CORRECTED MOMENTS

m = UNITS (b) PER CELL = 0.5

$\bar{X} = B + m\mu_1' = 5.500000 + 4.025150 = 9.474850$

$\sigma = m\mu_2'^{\frac{1}{2}} = 0.5 \times 1.597356 = 0.798678$

$K = \frac{\mu_3}{\mu_2^3} = \frac{-1.730023}{4.205178} = -0.4264870$

$B_2 = \frac{\mu_4}{\mu_2^2} = \frac{22.144232}{6.510389} = 3.401270$

$B_1 = K^2 = (-0.414470)^2 = 0.180175$

$\sigma_{\bar{X}} = \frac{\sigma}{\sqrt{n}} = \frac{0.798678}{\sqrt{15050}} = 0.006510134$

$\sigma_\sigma = \frac{\sigma}{\sqrt{2n}} = \frac{0.798678}{\sqrt{30100}} = 0.004603859$

$\sigma_K = \sqrt{\frac{6}{n}} \left(\sqrt{0.0265987}\right) = 0.0192667$

$\sigma_{B_2} = \sqrt{\frac{96}{n}} = \sqrt{0.0015968} = 0.0399334$

$3\sigma_{\bar{X}} = 0.0195310$

$3\sigma_\sigma = 0.0138105$

$3\sigma_K = 0.059001$

$3\sigma_{B_2} = 0.119800$

Reproduced from Figure 3 of paper on "Quality Control Charts" by W. A. Shewhart, "Bell Technical Journal", October, 1926.

$-0.7404 = \log h + 0.8451\,C$

$+0.3467 = \log h + 1.0416\,C$

$1.0871 = 0.1963\,C$

$C = \frac{1.0871}{0.1963} = 5.538$

$\log h = -0.7404 \text{ or } +0.3467$

$-4.6000 \quad -5.7879$

$-5.4204 \quad -5.4312$

$h = 0.0000037355$ Anar. $\log h = -5.4206$

$\frac{t}{15050} = 1 - (10)^{-0.0000037355\,R^{5.538}}$

Computation for values of t

R	2	3	4	5	6	7
log R	0.3010	0.4771	0.6021	0.6990	0.7782	0.8451
5.538 log R	1.6663	2.6422	3.3344	3.8711	4.3087	4.6805
log h	-5.4208	-5.4208	-5.4208	-5.4208	-5.4208	-5.4208
$\log\omega(-t)$	-3.7545	-2.7786	-2.0864	-1.5497	-1.1121	-0.7406
t %	00.06	00.38	01.22	03.55	16.33	34.19
R	8	9	10	11	12	13
log R	0.9031	0.9542	1.0000	1.0414	1.0792	1.1139
5.538 log R	5.0046	5.2846	5.5380	5.7673	5.9766	6.1688
log h	-5.4208	-5.4208	-5.4208	-5.4208	-5.4208	-5.4208
$\log\omega(-t)$	-0.4194	-0.1364	+0.1172	+0.3465	+0.5558	+0.7480
t %	38.06	61.40	85.10	96.00	99.97	

These tests and criticisms, are insufficient to arrive at a just estimate of the usefulness of any new tool or method.

The discussions by Mr. Tucker and Mr. Fisher are of special interest. The latter gives fair expression to the point of view and judgment of the professional statistician. Both express themselves in general terms, one without specific examples to support the statements, while the other is well supported with examples. Both make one criticism, which was undoubtedly valid. As stated by Mr. Tucker, it is that the precision of results by the graphical method is not indicated. Mr. Fisher puts it in a slightly different form and asks for formulas giving the probable errors of the constants in the equations.

By reference to Mr. Hall's discussion and to Fig. 37 with the description of its construction and purpose, it will be seen that Mr. Tucker's criticism has been met in a practical manner, while the formulas given by the writer* indicate a first approach to a mathematical determination.

Nothing need be added here as to the "danger involved", or as to the absolute necessity at times for the engineer to resort to "extrapolation". The fact that this process is facilitated by "straight line methods" is sufficiently proved by Mr. Hazen's illustration. Three of the four methods he compares make it possible to use straight lines for extrapolation with skew curves.

Mr. Tucker no doubt realizes that in small samples the true direction of the frequency curve is not necessarily defined exactly by the points at the ends of series of observations. He should also be aware that Pearson's equation and others, as well as the writer's, define theoretical functions, which are used to smooth out and approximate series of observations. As far as the writer is aware all the usual functions are continuous and there should be no criticism of Fig. 9 on that account. If one wishes to omit the dotted portion of the frequency curve, or the extension of the auxiliary curve below the value for $f(R) = 0.5$, it has no effect on the conclusions drawn from the curve.

As to Fig. 17 these data form the best set that could be found to illustrate the particular equation. On the other hand, the equation with those special values for the constants is not necessarily the best for the data. Mr. Tucker might have submitted better curves derived by any other methods, although it is doubtful whether this could have been done.

The writer disagrees with Mr. Tucker as to Conclusions 6 and 7. Engineering science has been gradually simplified and reduced to standard methods in many branches so that there are thousands of successful practitioners who do not have unusual training in mathematics. Statistical methods likewise should be so simplified and standardized that such men can make scientific investigations of frequency data for purposes of engineering design.

The book by Dr. R. A. Fisher previously referred to, appears to be a serious attempt on the part of an expert to be of such assistance. The writer continues to hope that his method also, may be of aid. Methods of procedure should be adjusted to the necessities of the particular case in hand. If warranted by the quality and character of the data available and by the use to which results are to be put, the most accurate and elaborate analytical methods

* See pp. 101–102.

of investigation should be used. On the other hand, if the results produced by the application of a little "skill and judgment" with a minimum of labor and effort, are at the same time of sufficient accuracy for the purpose, and commensurate with the extent and accuracy of the data, there is no good reason why skill and judgment should not be applied to the solution of a problem. Of course, the engineer has no desire to enter the field of the professional statistician in the investigation of problems in economics and insurance; at the same time the application of simplified statistical and probability methods may become of great assistance in the solution of engineering problems. If the statistician finds no advantage in such simplified methods for expediting certain classes of investigations, the Engineering Profession, nevertheless, may have added a new tool to its equipment.

Table 26 gives the fourth published solution by Mr. Arne Fisher for the problem of fitting a curve to certain data. This last solution is perhaps the quickest and easiest of any, although the χ^2 test shows that his solution by logarithmic transformation is slightly better. It is understood that the problem was used by Mr. Fisher as an exercise in least squares and to demonstrate the point that the Gram series could be used with such a skew distribution. In Table 23 (furnished by the writer) no criticism was intended on this score; in fact, the reference might just as well have been made to Mr. Fisher's solution by means of a logarithmic transformation.*

In view of this new solution, a more complete comparison of the various results is given in Table 33. The values of P as used in Pearson's test for "goodness of fit" show that, while Mr. Fisher's first two solutions (Nos. 2 and 3) are, as he says, excellent exercises, their value to a biologist is very doubtful. On the other hand, all the other methods, including the graphical solution, are equally usable to the biologist. While the variation in the value of P for these four solutions (Nos. 1, 4, 5, and 6) is not significant, the fact remains that the results from the graphical method without the use of an equation are shown to be reliable and to fit the observations almost perfectly. Extrapolation, the bugbear of the statistician and a regretted necessity to the enginer (and probably to the biologist as well), can be performed in this case, therefore, with great confidence by the purely graphical method.

Moreover, while Mr. Fisher's final solution (No. 5) was done very quickly, special tables were required, of which hardly one engineer in a thousand knows. The graphical solution can be accomplished with the means and experience possessed by all engineers, and the time required can also be measured in minutes.

Mr. Fisher also demonstrates the advantages of the graphical method in his Table 28. This excellent example of almost perfect curve fitting is completed with the aid of another special table of functions almost equally unfamiliar to engineers in the extended form necessary for such work. The writer cannot agree that these results fail to give any inkling about the frequencies of future rainfall. After several years of study and research no better guide has yet been demonstrated so far as he can learn, nor have the

* "The Mathematical Theory of Probabilities," by Arne Fisher, p. 260.

able discussions of this paper shown any better means, when based on the theory of probabilities. If Mr. Fisher could have shown that the annual rainfall which would probably not be exceeded more than once, say, in 500 years, as determined by any other method, was more reliable than the graphical estimate, his claim would have been established; but he proves that his estimate is not any more reliable. Hence, neither method has any advantage on this score. The choice must be based on different grounds.

TABLE 33.—Comparison of Frequency Determinations by Various Methods.

No.	Method.	Average variation.	Values of χ^2.	Values of P.
1	Pearson's results*	2.7	4.07	0.65
2	Fisher, Gram series, 5 terms†	0.6	36.44	0.00
3	Fisher, Gram series, 3 terms.‡	2.3	21.89	0.00
4	Fisher, logarithmic transformation.§	2.6	4.05	0.65
5	Fisher, Poisson-Charlier series.‖	1.8	5.68	0.35
6	Graphical solution, (total frequencies read from curve).¶	0.9	1.45	0.91

* "The Mathematical Theory of Probabilities," by Arne Fisher, p. 226.
† *Loc. cit.*, pp. 226–232.
‡ *Loc. cit.*, p. 232.
§ *Loc. cit.*, p. 260.
‖ See p. 83.
¶ See p. 89.

Table 29 presents another example wherein the conclusions to be drawn from the graphical and analytical investigations differ primarily because the two investigations are not conducted under the same conditions, unless the writer has misinterpreted the meaning of the expression, "the true presumptive curve". If the solid curve, Fig. 36, represents the result of combining all the forty samples, then this contention holds. Even if the curve is based on the first sample only, Mr. Fisher has evidently applied one condition in developing the curve which he omitted in the graphical solution. He omits the term in $\phi_3(x)$ in the series, to produce a symmetrical curve, while he makes no attempt to secure symmetry by the graphical method. If the complete data had been submitted even without the information that *a priori* considerations called for a symmetrical curve, it is quite possible that a curve derived graphically might have differed from the true presumptive curve as little as the curves of Fig. 26. With the *a priori* condition of symmetry added, there is no reason to expect a correct solution by the proposed graphical method when only one or two samples are used.

Messrs. Shewhart, Tucker, and Fisher all question the writer's statement that the graphical method was especially developed for the examination of comparatively small samples. Nevertheless, that continues to be the writer's position and he does not find any real evidence in the discussions to prove that the graphical methods are not just as reliable as others when applied to the same data under the same conditions. In fact, the only conclusion which he is disposed to withdraw is No. 3, which is not directly a problem in frequency distributions. The reason for failure in this application to population statistics is given in the discussion by Mr. Goodrich.

The literature and mathematics of the actuary are not familiar to the writer and, consequently, the general equation of Quiquet for the number of survivors in a mortality table was not previously known. Nor is it quite clear how his equation can be reduced to that of the writer, as it gives the values of a series of logarithms. Also, the quantity $f(\chi)$ appears as a factor in his principal term, while it is contained only in the exponent of the logarithmic base in the writer's equation, and does not appear in this exponent in Quiquet's equation. The writer has graduated one or two mortality tables and found that his equation of the duration curve can be used to give the number of survivors directly instead of the logarithm of the number.

Mr. Fisher's discussion contains many interesting and valuable suggestions. Among others this thought is raised: If the determination of a function, $f(x)$, which is distributed in some known manner although the variable itself is not so distributed, constitutes "a sound and vigorous theoretical foundation", then it may be possible to put the writer's equation on a more sound philosophical basis and remove it from the class of "empirical graduation formulas". The question then becomes, is it essential that the known distribution be normal, or may it not be approximately normal or even of some given skewness, so long as the distribution is always the same for this function? If this is admissible, then it becomes evident that the function, $f(x)$, which is linear on skew frequency paper, is always the same for any given value of the index, c (see the general form of the frequency equation as given by Mr. Goodrich on page 64).

Professor Pearson's test for "goodness of fit", mentioned more than once, is new to most engineers. It is used for testing the suitability of mathematical curves to describe frequency distributions.* If a series of observations has been divided into any number of classes with varying frequencies in each class (as was done by Mr. Shewhart with the data of Table 32), and a mathematical expression has been devised to describe the distribution, then the sufficiency of the expression as an hypothesis to account for the variation shown by the distribution, may be tested by the value of χ^2, which depends on the squares of the differences of the observed and computed frequencies distributed in each class into which the observations are divided. This quantity is defined as follows by Dr. Fisher:†

"If m is the number expected, and $m + x$ the number observed in any class, we calculate $\chi^2 = \Sigma\left(\frac{x^2}{m}\right)$ the summation extending over all classes. Here, m is the computed value and x the difference from the observed frequency. This formula gives the value of χ^2, and it is clear that the more closely the observed numbers agree with those expected, the smaller will χ^2 be."

The theoretical distribution of this quantity was determined by Professor Pearson and a table is given in Dr. Fisher's book for the practical application of the test. The writer has put the data of this table into graphical form, Fig. 39, which is sufficiently accurate for practical work. To use the diagram,

* The descriptions given are based on the book by Dr. R. A. Fisher, entitled "Statistical Methods for Research Workers in Biology," which was referred to in the discussion by Mr. Arne Fisher.

† "Statistical Methods for Research Workers in Biology," by R. A. Fisher, p. 77.

it is necessary to know the value of n which is defined as the number of "degrees of freedom" for the distribution. This is usually one less than the number of classes or groups into which the observations are divided. In Table 32, Column (10), there are 10 groups, hence $n = 9$. Entering the diagram with corresponding values of χ^2 and n, it is possible to determine the approximate value of P, if this lies between 0.01 and 0.9.

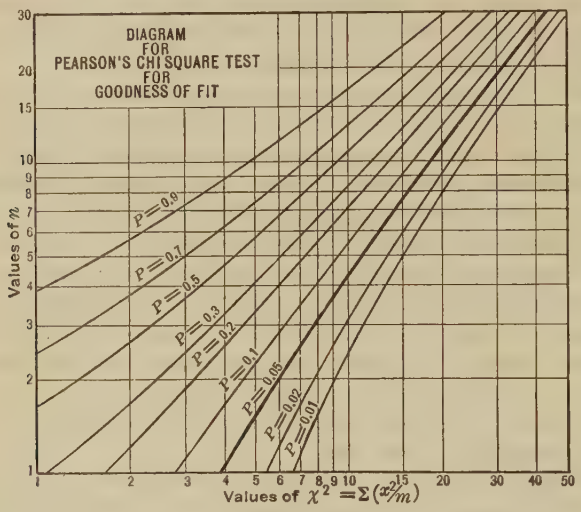

FIG. 39.

If there were a very large number of samples drawn from the same universe, but with slightly varying distribution in each sample, P would represent (theoretically) the proportion of the total number of samples for which the value of χ^2 would be greater than the value computed for the given sample. In Table 32, the values of χ^2 are all so large that if the curves were true definitions of the distribution of the universe, most samples should fit the curves more closely, giving smaller values of χ^2. On the other hand, the theoretical distributions in Table 28 fit the observed values so closely that P is more than 0.9, showing theoretically that random samples would usually give larger values of χ^2.*

In describing the use of his table, Dr. R. A. Fisher states that:

"If P is between 0.1 and 0.9 there is no reason to suspect the hypothesis tested. If it is below 0.02 it is strongly indicated that the hypothesis fails to account for the whole of the facts. We shall not often be astray if we draw a conventional line at 0.05, and consider that higher values of χ^2 indicate a real discrepancy".

Hence, in Fig. 39, the line for $P = 0.05$ is made heavy. When any given co-ordinates give values of P that lie to the right of this line, the fit may be regarded as unsatisfactory, while values on the left generally indicate satisfactory agreement between curve and observations.

* Mr. Arne Fisher states that the probable errors in the "skewness" and "excess" used in deriving his equation, are so large in this case that the values of P are not reliable.

In dividing the data into classes it is usually advisable to have several items in the first and last groups, as in Table 32. This was not done in computing the values of χ^2 for the comparisons of Table 33, as satisfactory agreement between curves and data was obtained in four of the six cases without special grouping.

The results of some additional studies since the paper was prepared, may prove useful in the application and further development of the methods. Equation (3) for the mode of Equation (B') is not in a form convenient for use; Fig. 40 is believed to be more easily applied in the determination of approximate values of this quantity. If z is put equal to the ratio, $\frac{R - a}{b - a}$, in Equation (2), this can be reduced to the form described in the discussion by Mr. Goodrich:

$$h = \frac{M}{c}\left(\frac{1 - z}{z}\right)^c \left(1 + \frac{2z - 1}{c}\right) \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

The curves in Fig. 40, plotted from Equation (12), may be used for finding approximate values of z when the values of c and h are given by the constants of a duration curve in the form of either Equation (B) or Equation (E). Having determined the value of z, the mode is found from the formula also given by Mr. Goodrich:

$$\text{Mode}_{BE} = z\,(b - a) + a \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

As stated by Mr. Goodrich (page 72), a graphical method has been devised for the theoretical determination of the limits used in the equations. As this method may sometimes prove valuable in the examination of large samples, and as some critics believe that the usefulness of the method is invalidated by the assumption of the limits, an explanation and example of this more complete procedure will be given. While it has been used with data that can be fitted by Equation (A), no actual series of observations suitable for this method has yet been found when Equation (B) is required, with its finite upper limit.

The method depends on the following relations. The general equation of the duration curve may be written:

$$n\,(10)^{-h\,[f\,(R)]^c} = n - t$$

Then,

$$h\,[f\,(R)]^c = -\log\left(1 - \frac{t}{n}\right)$$

When these values are substituted in the general equation for the frequency curve, it may be reduced to:

$$t' = \frac{c}{M}\frac{n - t}{R - a}\left[-\log\left(1 - \frac{t}{n}\right)\right]$$

for the case with limits, a and ∞, and to,

$$t' = \frac{c}{M}\frac{(b - a)\,(n - t)}{(R - a)\,(b - R)}\left[-\log\left(1 - \frac{t}{n}\right)\right]$$

for the case when both limits are finite.

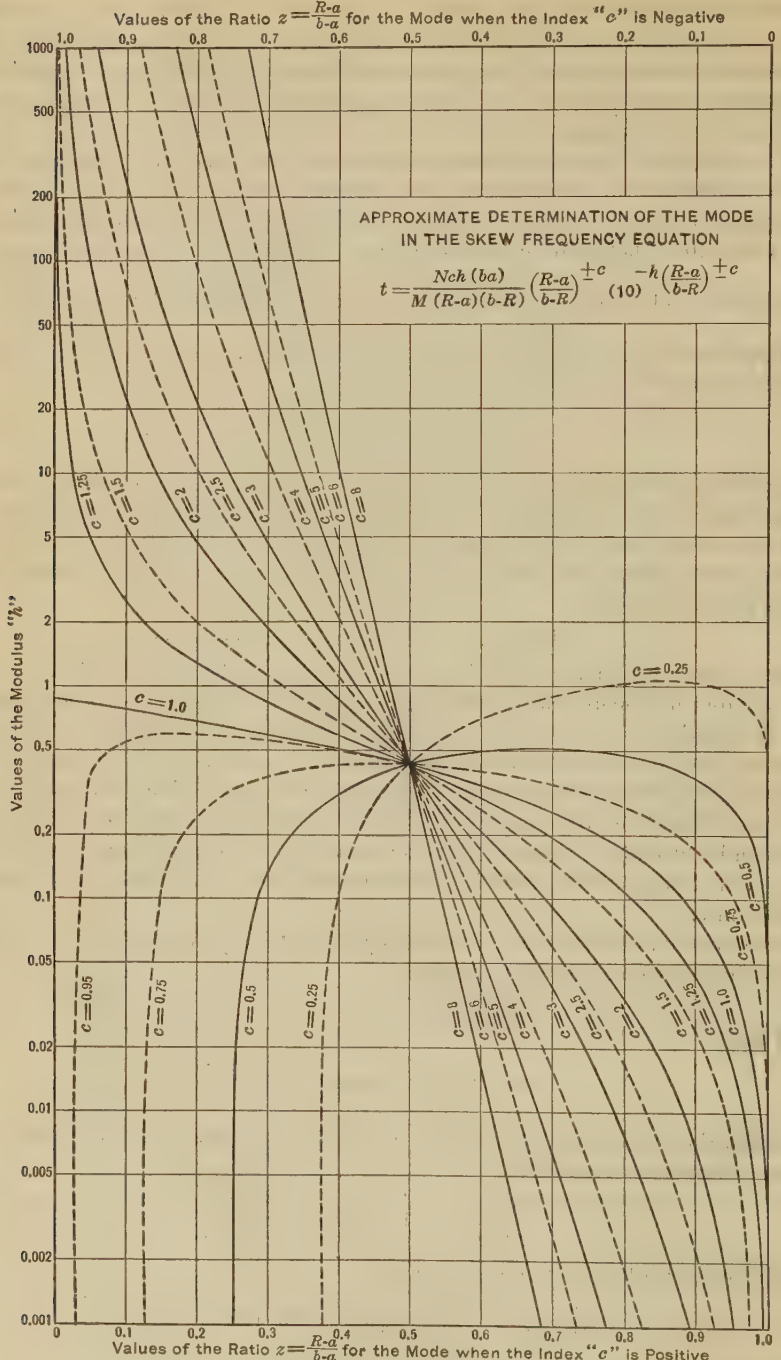
Values of the Ratio $z=\frac{R-a}{b-a}$ for the Mode when the Index "c" is Negative
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0
1000 500 200 100 50 20 10 5 2 1 0.5 0.2 0.1 0.05 0.02 0.01 0.005 0.002 0.001
Values of the Modulus "h"
APPROXIMATE DETERMINATION OF THE MODE
IN THE SKEW FREQUENCY EQUATION
$t=\frac{Nch\,(ba)}{M\,(R-a)(b-R)}\left(\frac{R-a}{b-R}\right)^{\pm c}{}_{(10)}{}^{-h\left(\frac{R-a}{b-R}\right)^{\pm c}}$
c=1.25 c=1.5 c=2 c=2.5 c=3 c=4 c=5 c=6 c=8
c=1.0
c=0.95 c=0.75 c=0.5 c=0.25
c=0.25 c=0.5 c=0.75 c=1.0 c=1.25 c=1.5 c=2 c=2.5 c=3 c=4 c=5 c=6 c=8
0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
Values of the Ratio $z=\frac{R-a}{b-a}$ for the Mode when the Index "c" is Positive

FIG. 40.

These relations may be re-arranged and written in the form of equations for a straight line and for a parabola of the second degree, thus:

$$Q = \frac{(n-t)\left[-\log\left(1-\frac{t}{n}\right)\right]}{t'} = \frac{M}{c}R - \frac{M}{c}a \quad \ldots\ldots\ldots\ldots (A_1')$$

$$Q = \frac{(n-t)\left[-\log\left(1-\frac{t}{n}\right)\right]}{t'} = \frac{-M}{c\,(b-a)}R^2 + \frac{M}{c\,(b-a)}(b+a)\,R - \frac{M}{c\,(b-a)}\,a\,b \quad \ldots\ldots\ldots\ldots (B_1')$$

Values of the factors in the quantity in the left-hand member of these equations can be computed from the data for a series of values of the variable, R; or approximate frequency, duration, and logarithmic curves can be drawn through points plotted from the data. When values of Q are plotted with the corresponding values of R, they should approximate a straight line in any case when Equation (A) will most nearly fit the distribution; and they will fall on a parabolic curve when Equation (B) is required. In the first case the value of the lower limit, a, can be determined from the slope and intercept of this line. The limit being thus given graphically, the values of $(R-a)$ may be plotted on skew frequency paper and the other constants of the equation may be found as previously described.

In the second case the equation is in the form:

$$Q = A\,R^2 + B\,R + C$$

in which,

$$A = \frac{-M}{c\,(b-a)};\quad B = \frac{+M}{c\,(b-a)}(b+a);\quad \text{and } C = \frac{-M}{c\,(b-a)}\,a\,b$$

If R_0 and Q_0 are corresponding values of the variables in this equation and X is put equal to $R - R_0$, with Y equal to $Q - Q_0$, it becomes possible to eliminate the constant term, C, and the expression may be written in the slope form of an equation of a straight line:

$$\frac{Y}{X} = (B + 2\,A\,R_0) + A\,X$$

The values of A and B, therefore, can be found from the slope and intercept of the line plotted with corresponding values of X and $\frac{Y}{X}$. With A and B substituted in the quadratic equation involving Q and R, a series of values of C can be computed and the average value of C determined.*

With the values of A, B, and C given, the values of the limits, a and b, are found by substitution in the formula:

$$\text{Limits} = \frac{1}{2}\left[-\frac{B}{A} \pm \sqrt{\left(\frac{B}{A}\right)^2 - 4\,\frac{C}{A}}\right]$$

* This method for the graphical determination of the constants of this quadratic equation is taken from Chapter I of "Empirical Formulas," by Prof. T. R. Running.

in which, the positive sign before the radical gives the upper limit, b, and the negative sign gives the lower limit, a.

A typical solution for the first case is given in Fig. 41, with the data and computations as shown in Table 34. The percentage frequencies were first computed and the stepped distribution curve was constructed. The trial frequency curve (dotted) was then drawn approximately through the centers of these steps, and the ordinates to this curve were read off and recorded in Column (7) of Table 34, for the given series of values of R. The items of

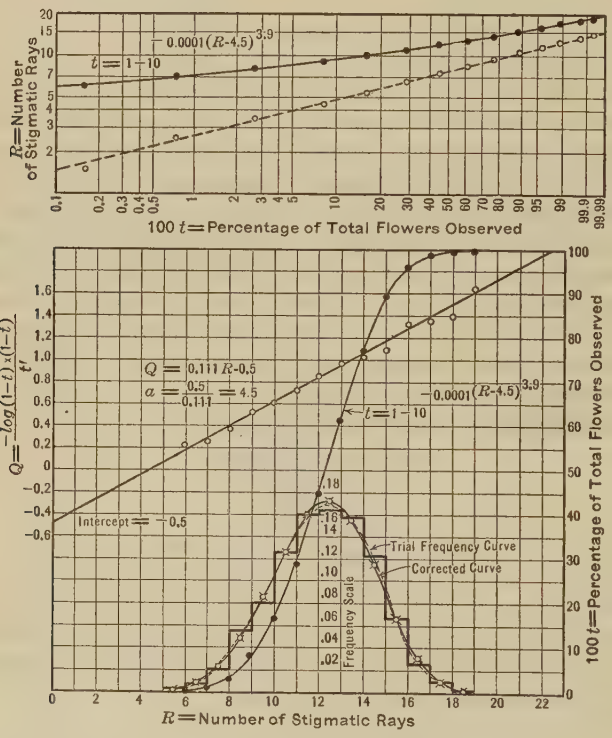

FIG. 41.

Column (3) were summoned progressively and set down in Column (4) and Columns (5) and (6) could then be made up. With the values of the quantities in Columns (5), (6), and (7), the values of Q, Column (8), were computed by slide-rule, and the points shown by the small circles were plotted with rectangular co-ordinates for the first auxiliary curve. The equation of this was found to be:

$$Q = 0.111\,R - 0.5$$

from which the lower limit, $a = 4.5$, was computed. With this, the values of $f(R)$ were computed, and the usual curves were then plotted on skew frequency paper as shown in the upper part of Fig. 41.

The equation of the curve was finally determined and the theoretical frequencies were computed and compared with the original distribution as

TABLE 34.—Distribution of Stigmatic Rays of Shirley Poppies.*

Number of rays, R.	Capsule distribution.	Percentage distribution.	Results from Trial Curve.						Computed Quantities.				
			t.	$1-t$.	$-\log(1-t)$.	t'.	Q.	$R-4.5$.	t.	Distribution. Percentage.	Distribution. m.	Difference, x.	$\frac{x^2}{m}$.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
6	3	0.16	0.0016	0.9984	0.0007	0.003	0.233	1.5	0.0011	0.11	2.1	0.9	0.39
7	11	0.58	0.0074	0.9926	0.0032	0.012	0.265	2.5	0.0081	0.70	13.3	— 2.3	0.40
8	38	2.00	0.0247	0.9726	0.0121	0.033	0.357	3.5	0.0300	2.19	41.7	— 3.7	0.33
9	106	5.56	0.0830	0.9170	0.0376	0.066	0.522	4.5	0.0780	4.80	91.4	14.6	2.33
10	152	7.98	0.1628	0.8372	0.0772	0.105	0.616	5.5	0.1628	8.48	161.5	— 9.5	0.56
11	238	12.50	0.2878	0.7122	0.1474	0.143	0.721	6.5	0.2888	12.60	240.0	— 2.0	0.02
12	305	16.00	0.4478	0.5522	0.2579	0.166	0.858	7.5	0.4488	16.00	304.8	0.2	0.00
13	315	16.53	0.6131	0.3869	0.4124	0.165	0.967	8.5	0.6214	17.26	328.8	—13.8	0.58
14	302	15.86	0.7717	0.2283	0.6415	0.142	1.031	9.5	0.7763	15.49	295.1	6.9	0.16
15	234	12.28	0.8945	0.1055	0.9768	0.092	1.096	10.5	0.8906	11.44	217.9	16.1	1.19
16	128	6.72	0.9617	0.0383	1.4168	0.041	1.324	11.5	0.9573	6.67	127.1	0.9	0.01
17	50	2.62	0.9879	0.0121	1.9172	0.017	1.365	12.5	0.9872	2.99	57.0	— 7.0	0.86
18	19	1.00	0.9979	0.0021	2.6778	0.004	1.405	13.5	0.9973	1.01	19.2	— 0.2	0.00
19	3	0.16	0.9995	0.0005	3.3010	0.001	1.651	14.5	0.9996	0.23	4.4	— 1.4	0.45
20	1	0.05	1.0000	0.0000	.....	.....	.....	15.5	1.0000	0.04	0.8	0.2	0.05
Total.....	1 905	100.00	.....	.....	.....	.....	.....	....	.....	.....	.....	Av. = 5.3	$\chi^2 = 7.33$ $P = 0.92$

First Auxiliary Curve:

Slope $= \frac{M}{c} = 0.111$

Intercept $= -\frac{a\,M}{c} = -0.5$

Lower limit $= \frac{0.5}{0.111} = 4.5$

Second Auxiliary Curve:
Slope $= c = 3.9$
$h = 0.0001$

Equation:
$t = 1\ (10)^{-0.0001\ (R-4.5)^{3.9}}$

* Data from Table III, p. 78, "Introduction to the Theory of Statistics", Yule.

shown in Table 34. (The distribution is so nearly symmetrical that a normal curve might have been fitted to the data, probably with equally good results.)

While satisfactory curves can usually be obtained more quickly by the method of trial and error, it is evident that, when suitable data are available, all the constants in Equations (A), (B), (D), and (E), can be determined by graphical methods. With Equations (D) and (E) values of $\frac{t}{n}$ and $-\log\left(\frac{t}{n}\right)$ are used instead of the corresponding quantities in the foregoing analysis. It is evidently possible also to apply analytical methods throughout. It is seen from this, that equations derived by this graphical method depend on three curves of the first and second orders only, while skew curves of the Pearson types depend on curves of the first, second, third, and, sometimes, the fourth order. As the probable errors of the parameters of these curves seem to increase with the order of the curves on which they depend, it may be that there is a sound basis for the remarkably good fitting of curves by graphical methods as compared with those deduced by analytical processes.

It is hoped that a little more confidence may be placed in these graphical methods of examination and analysis of skew frequency data after a careful consideration of all the discussions and the explanations of the various points raised. It is only by such frank discussion that the strength and weakness of a new method of investigation can be brought out and errors or misunderstandings corrected. The writer feels well repaid for his efforts and expresses his sincere thanks for the many valuable suggestions contained in the several discussions, and his deep appreciation of the wide interest shown.

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Paper No. 1623

SNOW REMOVAL PROBLEMS OF ORGANIZATION AND OPERATION*

By George E. Hamlin,† M. Am. Soc. C. E.

With Discussion by Messrs. Howard C. Means, Z. E. Sevison, B. J. Finch, W. A. Van Duzer, and George E. Hamlin.

The operation of snow removal, like construction or maintenance, must be approached from the viewpoint of organization or personnel as well as from that of equipment, and to this personnel the greatest attention must be given. It is only by reason of a thoroughly competent, efficient, and loyal organization that satisfactory results can be produced in the limited time allowed for the opening to operation of the main and connecting road systems under the charge of a State Highway Department if travel is to be continued with the minimum inconvenience.

Through its very nature, long hours, extreme weather conditions, and personal discomfort enter largely into the work. It is only the desire of the organization to fulfill its function and its "not-to-be-beaten-at-any-cost" attitude that will cause the men to "carry on" through day and night, through storm and zero weather, continuously if necessary, until the sections under their charge are free for travel.

The experience of several years in the development of a snow removal organization in Connecticut has been utilized in the general recommendations for this maintenance problem.

In planning a snow removal campaign, it must be considered that owing to the general lack of surplus maintenance funds, only average conditions can be considered and planned for. The purchase of equipment alone represents the expenditure of many thousands of dollars and in the average locality, such

* Presented at the meeting of the Highway Division, Salt Lake City, Utah, July 9, 1925.
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equipment also must be suitable for types of work other than snow removal. Thus, the highest type of snow-fighting equipment cannot always be economically furnished.

The personnel must be largely the same as that employed in yearly maintenance with the addition of units required to operate efficiently under the extreme conditions which may occur. Each foreman should be in touch with additional men who, in the case of urgent need, will be used to reinforce the regular patrol force or replace it when continuous operation becomes necessary.

To the regular organization, however, must be delegated the brunt of the work. These are the men who are familiar with both summer and winter conditions in their districts. Each landmark along the road is known to them almost as if it were in their own dooryard. A tendency for drifts to form under certain wind conditions is remembered and measures are taken to counteract this. Each foreman is certain of the ability of his crew to operate more quickly and efficiently than the crew in the adjoining section and this feeling is largely held by the men themselves. It is their section, under their charge in winter as well as in summer—"The next gang has never beaten us yet! Come on, boys!" Where this feeling exists the larger part of the personnel problem has been solved.

The regular maintenance organization for carrying on snow removal will include:

1.—A Maintenance Engineer and Assistants at Headquarters;
2.—A District Maintenance Engineer in charge of a Supervision District; and
3.—Foremen and laborers on each Patrol Section.

The Maintenance Engineer is in full charge of the snow removal program. During the progress of clearing snow, he is necessarily confined wholly to his office in order to receive reports and to give instructions incident to the changing conditions or the severity of the storm in any particular locality. To him each District Maintenance Engineer reports by telephone regularly during the storm, giving the location and condition of equipment, the progress of snow removal, and recommendations as to transfer of additional equipment.

On his desk is a map of the State and as each District Engineer reports, the information is transferred graphically to the map, making it an accurate progress chart. On this map is also recorded additional information reported by the District Engineer, including the depth of snow and the location of drifts. Ordinarily, two operations of a plow will clear the roadway to a sufficient width to permit the passing of traffic. On the map, a single line denotes that a single lane has been opened. After the next passage of the plow an additional line is added to represent the opening of the road to two-way traffic. All roads are opened to this width before removing any snow from the shoulders and gutters, in order to open up waterways and provide room for the snow of future storms.

The Assistant Maintenance Engineers function directly under the Maintenance Engineer. They inspect the equipment and the progress of the work and keep in frequent touch by telephone with the main office. Each of these

men has a State-wide knowledge of conditions and is fully capable of taking complete charge in emergency, thus relieving and supplementing the District Maintenance Engineer in a snow-congested district.

The Foremen report by telephone as frequently as possible to the District Engineer. This is necessary, not only to have information available for the traveling public, but to permit the transfer of any particular organization unit, under necessity, to a point where conditions require immediate attention.

A chart in the main office also carries the locations of each of the snow removal units, these being represented by buttons inscribed with the type and character of such unit. When particularly adverse conditions are reported from a certain district, or a snow removal unit goes out of commission, which sometimes will occur when many of such units are in operation, an examination of the location chart in combination with the progress chart will determine the best method of transfer. Orders to the District Engineer are telephoned and frequently within ½ hour the relief unit is on its way to the new location. The merchant or property owner with a telephone connection along the ploy route has proven invaluable in getting in touch with the foreman in the shortest time.

The necessary equipment will vary largely with the location and average climatic conditions. In Southern New England, the maintenance trucks are utilized for most of the snow removal. Each truck of 2 tons capacity or more is fitted with a push plow with a 10-ft. blade, is in charge of the regular foreman, and is assigned to the section which is ordinarily patroled and maintained.

The trucks usually start operating when the snow has reached the depth of 2 to 3 in. and are used continuously until the roads are cleared, sometimes for a period of 36 hours or more. The trunk line roads have first place in the program, being followed by the State Aid or secondary roads.

Each truck carries from two to six laborers, depending on the severity of the storm, and the necessity for shoveling at some particular location. In hilly sections, where more snow can be anticipated, tractors fitted with V-type plows are utilized in addition to the truck equipment. These tractors must be designed for mobile use so that they may be transferred to the location of the greatest depth of snow during any particular storm.

Plow equipment should be properly located for convenience of operation, so that no time need be lost by necessity of distance of travel. Average snow conditions will enter largely into this location.

Familiarity with the section to be kept open is one of the greatest assets in snow removal. During a blinding snowstorm with the temperature near the zero mark, it may be difficult even to locate the highway. The foreman who is in charge of this district throughout the year will often recognize topographical features and thus be enabled to minimize delays.

It is desirable to furnish each District Engineer with one or more additional plows and a supply of parts to be transported by light truck on telephone notification. Where possible, it is also advantageous to keep in reserve one or two trucks in each district to replace damaged equipment. A small reserve of men for emergency use is provided by the foreman.

In Connecticut, a total of 145 trucks are fitted with snow plows to clear about 1 800 miles of highway, an average of 13 miles per truck. This average, however, does not in any way represent the mileage of any particular truck, which is governed wholly by the importance of the road and the conditions in each locality. On the heavily traveled main thoroughfares between large cities a truck may be scheduled for less than 5 miles of plowing, while on the less important highways and connecting State Aid roads, the distance may be 20 miles or more.

The work of each truck is scheduled in accordance with the importance and necessity of traffic. On some sections of the main Boston-New York highway plows are worked in batteries, as in this way mechanical trouble occurring to one truck does not necessarily tie up any particular section of this important highway for a considerable period.

Mobility of equipment is secondary only to personnel. From telephone reports received from the foremen through the District Engineer, to the Maintenance Engineer, information is rapidly developed in case of emergency, so that equipment may be transferred and centered at any particularly congested location.

During the three seasons, 1922-25, the cost of snow removal in the State of Connecticut has been as given in Table 1.

TABLE 1.

Year.	Total cost.	Average cost per mile.	Mileage cleared.
1922-23	$175 021.78	$116.07	1 500
1923-24	53 800.20	30.35	1 800
1924-25	70 453.86	39.75	1 800

During the winter of 1924-25, while the average cost for the State was $39.75 per mile, an analysis shows a cost of $15.42 per mile in a district located in the southeastern part of the State adjoining Long Island Sound, as compared with $95.32 per mile in the northwestern section in which the lower Berkshire Hills furnish more severe weather conditions.

For several years Connecticut has erected snow fences in exposed locations where drifting conditions can be expected. Usually these justify their cost and hence the mileage has been increased each year. The location should be carefully chosen in accordance with prevailing wind direction, otherwise snow may be banked in the highway by a contrary wind and travel blocked at this point. By the removal of brush along the sides of a highway, frequently the tendency for drift forming can be eliminated. Experiments will often determine the solution at minimum expense.

Each Maintenance Engineer is suffering with the complaint of all maintenance engineers—lack of equipment beyond ordinary requirements. A severe snowstorm with an accompanying high wind will develop this complaint almost on the moment into an acute attack requiring immediate opera-

tion. This is the danger point. The crisis always passes rapidly. There may remain local difficulties to be taken care of, but during the height of the attack these may be forgotten. Their solution will follow in natural course when traffic is rolling with the least inconvenience over the length of improved highways in the State.

The investment in such highways represents one of the State's greatest business assets. The inconvenience and expense connected with their non-use even for a short period are enormous. Snow removal must be a definite part of each highway program if the maximum value from the investment is to be obtained.

DISCUSSION

HOWARD C. MEANS,* M. AM. SOC. C. E.—The organization and operation of snow removal have been ably detailed by Mr. Hamlin, but it is a question just how many States work under these ideal conditions. The removal of snow from road surfaces is one of Utah's newer maintenance problems. In the days of horse-drawn vehicles no one ever thought of snow removal except for pedestrians. In the early days of automobiles the owner generally housed his car during the winter or at least confined it to town use. With the present tremendous investment in motor cars, together with the absolute necessity of using them in all walks of life, however, the economic demand is such that main trunk-line highways must be kept open for constant use, winter and summer. This demand justifies the expenditure of almost any amount of money within reason.

Another reason for the constant use of the highways during the winter is the extensive use of non-freezing solutions in automobile radiators and the increased number of closed cars. This makes travel by automobile both possible and comfortable. Within a few years snow removal will be regarded as an essential maintenance operation, as railroads look on it to-day. Even now the car owner who happens to be held up for a short time on a road that is being kept open has cultivated the habit of complaining about the delay instead of being happy that he can get through at all. The problem of clearing road surfaces of snow in the most economical manner is confronting the highway maintenance organization of all States where snowfall occurs.

In addition to these economic features of winter travel, there are three reasons why highways should be cleared:

First.—If snow remains on the road surface a layer of ice forms, and the action of tire chains soon cuts ruts through this ice to the pavement. Cars are compelled to follow this one lane of travel, as it is next to impossible to leave it when the occasion requires. This condition caused many accidents in Utah during the winter of 1924-25 in sections where it was impossible to commence clearing the snow immediately after it fell. On one inspection trip between Salt Lake City and Nephi, a distance of 90 miles, five cars were reported with bottoms up.

Second.—The result of confining traffic to these single lanes shows a tremendous wear on the paved surface. In the case of one bituminous pavement, laid during the previous fall, it was found after the surface had been cleared that the action of the tire chains had torn loose considerable of the wearing surface and had formed ruts throughout its entire length.

Third.—Spring maintenance is naturally increased when snow is left on the road surface during the winter. Melting snow, confined as it is, soon soaks the sub-grade and produces disastrous results.

There are several points in Mr. Hamlin's paper that should be emphasized. Of course, organization and equipment are essential in all lines of endeavor connected with highway construction and maintenance, but they

* With E. B. Palmer Co., Salt Lake City, Utah.

are more essential in snow removal than perhaps in any other kind of work, on account of the conditions under which an organization labors and the kind of tools with which it has to work. As an illustration, during the winter of 1924-25, in the removal of snow on one section of road in the northern part of Utah, it was necessary to keep in constant use the available equipment on account of a stiff wind that filled the roadway with snow about as fast as it could be cleared. The low temperature, about 37° Fahr. below zero, necessitated the use of a distillate in the radiator, as no anti-freezing solution could be used successfully.

By "organization" is meant a crew of men that can be depended upon to perform their work regardless of time or conditions. The very nature of the job requires loyalty, untiring efforts, and courage to grapple with work that has the appearance of approaching the impossible. These crews are called upon to work long hours, night and day, because it has been demonstrated that the time to commence the work is when the snow begins to fall. In the mountain States the high passes required to be kept open are naturally the most difficult sections. In Utah, the winter of 1924-25 was the first in which an organized effort was made to remove the snow from the main highways.

It was found possible to keep open the road from the Idaho line on the north through the entire length of the State to the Arizona line on the south. No attempt was made to clear the east and west highways. The Uintah Basin in the northeastern part of Utah is without rail transportation, consequently it was necessary to keep open this connecting link over a summit 9 200 ft. high. This was accomplished by working three 10-ton caterpillar tractors each equipped with a snow plow, the work being carried on in co-operation with the U. S. Postal Service.

The kind of equipment to be used in snow removal is a question that will bear a great deal of observation and study. Throughout the highways where only a light snowfall occurs the problem is comparatively easy, the snow being pushed off by plows attached to trucks and tractors. By constant attention the highways can be kept clear with a reasonable amount of effort. In high country, however, the problem is not so easy. Where snow falls each winter to a depth of perhaps 5 to 8 ft., and the wind blows almost constantly, it is a difficult problem. Through one pass in the northern end of Utah, this was accomplished by the use of caterpillar tractors and snow plows with blades built up to the height of 5½ ft. However, with this kind of equipment it was found that the snow quickly piled up along the sides. It was a tremendous task to shove it off the grade or dispose of it in some way so that new snow could be handled. With rotary plows the snow might be removed entirely with one handling.

Another usable piece of equipment for removing snow to a depth of 12 in., is the Hadfield-Penfield or Weir one-man grader, using a Fordson tractor as the mobile unit. The advantage of this equipment is that it can move quickly and cover considerable distance, but of course it is naturally confined to light work.

The usual guard-rail necessary during the summer on high mountain roads becomes a serious menace during the winter and considerable thought has been expended in just how to devise the removal of guard-rails. Possibly, this can be accomplished by sawing off the wooden posts close to the ground and using a 2-in. pipe as a dowel connection between the stationary and movable parts. Snow fences perform an important part in some sections of highways in this mountainous country. Usually the prevailing winds are from one direction and the problem of placing snow fences in such a position as to hold the snow successfully is a question requiring individual study. The necessity of considering the snowfall and the direction of wind in the original location of highways is an important problem of the locating engineer. Excessive construction costs are not justified in attempting to arrive at an ideal condition, but often locations where the direction of the prevailing winds will clear the snow from the road surface without additional cost may be chosen. In other cases the location is made after exhaustive study, with the definite knowledge that snow removal is to be a requirement.

During the winter of 1924-25, snow removal was carried on in Utah along several hundred miles of road. Much of this presented no particular difficulty as it was in a country of light snow. However, 134 miles did present a real problem. The cost of this work averaged $81 per mile, ranging from $456 per mile on one section of mountainous road to as low as $15 per mile on another section adjoining it.

Where snow removal is necessary the cost should be a part of the annual maintenance budget and should be approached with as definite an idea of accomplishment as every other part of maintenance.

Z. E. Sevison,* M. Am. Soc. C. E.—The speaker desires to present those principles adopted at the very beginning of the construction of the highway system of Wyoming which have governed the location, design, and construction of all such highways built in that State. It is believed that these principles are worthy of careful consideration in all localities where snow conditions prevail, in order that this hazard may be reduced to a minimum.

The proper location is of primary importance and should be governed by the topography, the prevailing direction of the wind during the winter, and the exposure, with grade and curvature, within allowable limits, receiving secondary consideration. The sacrifice of alignment and grade to a reasonable extent may be necessary to secure a snow-free road. In a country subject to snowfall, the successful locating engineer must not only be familiar with the territory in which he is working, but should have a good knowledge of the winter weather conditions which generally prevail.

In the open areas of the Western States the location should be made on ridges or on slopes against the prevailing direction of the winter wind, so that an elevated roadbed will be swept clear. All cuts are to be avoided, but where this is impossible the location and construction should be such that they will be swept clear by the wind. In the mountainous areas where

* State Highway Engr. of Wyoming, Cheyenne, Wyo.

the traffic is not sufficient to justify keeping a road open during the winter months, proper exposure is fundamental, and in conformity therewith the best possible grade and alignment that the country affords should be secured.

The winter season may be utilized to advantage in determining the location except in forests or mountains, or a preliminary study may be made with sufficient notes as to snow conditions so that the locating engineer may be governed by these when the survey is made. If neither of these plans is followed the final inspection of the line should be made in the winter and prior to the completion of the plans. The notes of the location survey must be complete as to snow conditions so that the designer will have all the necessary data.

The design of the highway from the standpoint of snow hazard must be in the hands of an experienced engineer. In general, the surface of the roadway should be from 1 ft. to 3 ft. above the ground line at center line. This is governed by the lateral slope, vegetation, and the drainage requirements. Where cuts are unavoidable they should be "daylighted" or widened to the right-of-way line and all material used in fills or wasted below the grade line. The elevated grade line has the advantage of simplifying snow removal, should that be necessary, and this practice should commend itself to the Eastern highway engineer; it also provides better drainage. In addition, some cuts will require snow fences. A right-of-way width of not less than 100 ft. is recommended in order to reduce the possibility of drifts caused by weeds along the right-of-way fences, and to furnish ample borrow.

Through forests and mountainous country where it is improbable that traffic requirements will demand a year-around road, or where topographical conditions govern, the usual design of balanced quantities should be modified to the extent of elevating the grade line sufficiently so that every advantage of correct exposure will be utilized and a dry roadbed obtained at the earliest possible date each season. This is important for the reason that few mountain or forest projects have been surfaced, principally due to lack of funds. The construction requirements are practically the same as for the usual location and design. All vegetation, brush, sage brush, and trees should be removed from the right of way and borrow-pits should be located so as to take advantage of slopes and wind.

It is realized that highway location and design as herein described may not appear adaptable in the older sections of the United States where the territory is closely built up and it may be necessary to follow established roads and grades, and where rights of way are narrow, yet the speaker is convinced that these principles may be closely approached. The major difference between the East and West is characterized by a heavy snowfall in the East without much wind, while in the West (Wyoming particularly) the snowfall in any one storm is usually light, accompanied by winds which cause drifting. These drifts become hard-packed within the period of duration of the storm and are difficult to remove with ordinary equipment. Even weeds or grass along the shoulder, or a right-of-way fence close to the roadway, will cause drifts, hence

the importance of proper maintenance to insure clean shoulders in the fall of the year, and wide rights of way.

When the traffic has reached a volume on an old road such that reconstruction becomes necessary or a parallel route built, the cost of snow removal on the old route should be compared with that on a route properly located and designed. It seems probable that this item of the final cost of a road has not been given the consideration it should have had in arriving at the economic location.

B. J. FINCH,* ESQ.—Mr. Hamlin has thoroughly covered the problems of organization and operation in snow removal. These problems are of a general nature and apply as well to the sparsely settled regions of the West as to the densely populated sections of the East. Snow removal is so closely related to maintenance that it is difficult to separate the two.

An organization for snow removal necessarily has its nucleus in the regular maintenance forces. Because of the variation in the time at which and for which the extra personnel is needed, there must be a great measure of elasticity in the make-up of the forces. This means that the regular force of the maintenance department must be well trained, loyal, and capable of functioning within itself as well as when greatly expanded.

One of the features of snow removal work which applies more peculiarly to the mountainous regions is that of opening the mountain passes in the spring after the snow has disappeared below and only the passes prevent communication between the valleys. This applies to transcontinental roads as well as to local roads, as wherever the passes in the intermountain region are above 8 000 ft. elevation, all communication except by sleds becomes dangerous and almost impossible for a period of from three to five months.

When the time comes for opening these passes, there are two distinct problems to be solved. The first is, of course, to get the road opened in the most economical manner and in the shortest time. The other is to protect the grade and structures from erosion while the snow is melting; this can be done only by the prevention of washing by patrolmen who direct the flow of water to the regular channels as closely behind the melting snow as possible.

Many different devices are being proposed for this type of snow removal, from the heavy rotary plow to the simpler forms of grader. The amount which can be spent for opening a road depends on its use. If a road is of sufficient importance so that its opening a month in advance of the regular season is worth considerable to the traveling public, a proportionally great expense is justified. Where the traffic is small, however, say, below 50 cars per day, cheaper methods of removal are necessary even if the time of opening is delayed somewhat.

One method which is being tried with success in many sections where a fair proportion of sunshiny days prevails is that of "dusting" or "sanding" the snow. This operation may be briefly described as sprinkling on the surface of the snow a dark colored, light weight, granular material which, when

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exposed to the sun's rays, attracts and holds enough of the heat to cause the snow around the particles to melt. This breaks up the even surface of the crust and greatly hastens the disappearance of the drifts. This method is being tried with good success in some parts of Idaho. The work of "sanding" or "dusting" can be done by the same patrolmen who are looking after the flow of water in the ditches. This method often reduces the time for opening a pass to half that otherwise possible without a heavy expenditure. It also makes unnecessary the shoveling which otherwise would be necessary to eliminate many drifts that may be cut away by the action of the sun. A crew of two or three men can do the work of a much larger outfit, and the organization correspondingly reduced.

Another factor that enters largely into the question of snow removal is the location and design of the roads. In the densely settled sections of the East the location of a road is more or less fixed by improved real estate. In the West, however, there is opportunity, particularly in the roads over the mountain passes, to locate the new highway in the place where interference from snow will be a minimum. In some cases snow studies have extended over a period of several winters, with the result that when the road has been built it lies where drifting is least likely and where the exposure is such that most of the snow removal is done by the wind and sun.

The day when main highways can remain closed for months is past. The public is demanding, as it has a right to do, traffic facilities for continuous use of the millions of motor cars which it has purchased. Engineers must face this situation squarely and organize their maintenance departments so that the roads will be in acceptable condition not only for the season of heavy travel, but also for the lighter, but possibly more necessary, traffic during the winter months.

W. A. Van Duzer,* M. Am. Soc. C. E. (by letter).—Snow removal, to a greater extent than any other highway activity, is affected by local conditions. In fact, generalization is difficult in dealing with this topic, since the work in one locality or State is likely to be so different from that in another.

The scope and importance of snow removal are determined chiefly by the amount of snowfall, which measures the initial level depth to be disposed of; the extent of drifting, which may increase the depth by 40% and which aggravates the removal problems and necessitates the handling of quantities in excess of the level-depth snowfall; and the volume of traffic, which to a considerable extent determines the necessity of, and the justifiable expenditures for, snow removal.

An idea of the amount of snowfall can be obtained from annual averages as reported by the United States Weather Bureau. In considering averages, however, it is necessary to bear in mind that they change from year to year, and in any one year a State average may represent wide variations. For instance, the annual snowfall for the State of Pennsylvania for the winter of 1923-24 averaged 48.0 in.; for the winter of 1924-25, 43.1 in.; and for the winter of 1925-26, 50.4 in. The 1925-26 average was computed from forty-nine

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stations ranging from a minimum of 17 in. in the southeast corner of the State to a maximum of 137 in. in the northwest corner.

The extent of drifting is more difficult to visualize. There are so many variable factors that for future reference the safest way to study causes of drifting is to go out on the road itself after the drifts have formed. Even then, it must be considered that in the next storm the wind may come from a different direction and create different drifts. A fair realization of the possibilities of drifting, however, can be obtained from a study of a chart showing the topography, direction, and force of prevailing winter winds, and the monthly mean temperatures. The topography will indicate the extent of grading, in which cuts and side-hill sections are significant; the direction and force of prevailing winds will show which roads will catch most of the cross-drifting and how serious the drifting is likely to be; and the temperature data will indicate, to some degree, how long the snow may be expected to lie along the road, without crusting and dry enough to continue subject to drifting. Greater care is now being exercised in the location of highways to minimize the danger of drifts.

The volume of traffic naturally varies with the density of population adjacent to the road, and, therefore, in the absence of traffic-flow data, if the snow-removal conditions of one section are known, the indices of density of population will furnish a good basis of comparison for any other section.

Although fundamental conditions present such wide variations, there are two principal problems which concern all States in which any considerable snow removal is to be undertaken. The first of these is to keep the road open continuously. It would be comparatively easy to remove snow if there were no time element. It is the necessity of removing the snow before it has accumulated to such an extent as to block traffic that makes snow removal a serious and costly proposition. In Pennsylvania, all roads are considered open for travel by the State office; and information is given to the public unless advised by the field that the road is closed, in which case an estimate of the time the road will be closed is included in the report.

The second is the always present problem of costs. It is necessary to hold the cost of the work down to a minimum. At the same time, it is necessary to make the accounting complete so that other lines of activities are not inequitably burdened with charges. The State of Pennsylvania during the past years has been practically subsidizing snow removal by charging truck and tractor equipment at the same rental rates as on other road work. It has become necessary to increase these rates by an average of 60% for snow-removal work on account of the heavy repairs and depreciation that attend the use of tractors and trucks in plowing snow.

Neither of these two problems can be considered separately; both resolve themselves into one problem of efficiency. Viewing snow removal in this way, several points present themselves, such as program, organization, location, snow fence, equipment, personnel, operations, and costs.

Program.—The first step in snow removal is laying out the program. This must be done with due regard both to the budget and to traffic requirements.

The most readily apparent test of the economics of snow removal is from the standpoint of the protection of investment. The fixed charges, depreciation and interest, on a high type of surfaced road, for instance, will be about $4 000 per mile per year, or more than $10 per mile per day. If winter and summer density of traffic were assumed to be equal, it would be economical to spend for snow removal as much as $10 per mile for each day that the road would otherwise be blocked. Or, assuming the winter density of traffic to be normally one-half of the annual average, the justifiable expenditure, from the same standpoint of protecting investment, would be $5 per mile for each day that snow would block the road. Under the second assumption, in a locality where snow would block the road 60 days a year, an expenditure of less than $300 for snow removal, if it kept the roads continuously open, would evidently be advantageous.

The same line of reasoning applied to lower types of hard surfaced roads, representing smaller first cost, would, of course, produce lower figures, perhaps as low as $1.50 per day; but in the case of these roads, there is an additional factor to be taken into account. Winter snow removal undoubtedly lowers the cost of spring repairs. Therefore, it would be fair to group all roads that carry an annual daily average of more than 500 vehicles and assume for them the same value of snow removal as for the high type surfaces.

Whether or not the investments and saving in repair costs are considered, the final check on the snow-removal program is to see if it is within the budget, if it is practicable from the standpoint of forces and equipment available, and if the mileage is laid out to best serve the localities and the traffic. Snow removal in Pennsylvania costs about 40 cents per motor vehicle registered.

Organization.—The organization for snow removal should be designed to cope with the particular conditions it will have to handle. Mr. Hamlin's description of such an organization indicates a highly centralized control. Under the conditions with which the writer is familiar, the work is such as to require decentralization of control for efficiency.

The Pennsylvania program comprised more than 5 000 miles during the winter of 1925-26, and about 6 000 miles for the winter of 1926-27. Some of the work is more than 300 miles distant from headquarters, so that the plan of organization to insure unimpeded operation is:

1.—Central headquarters at Harrisburg.
2.—The State divided into four Engineering Divisions, approximately the northwest, northeast, southwest, and southeast quarters of the State; a Division Engineer resident in each, reporting to Harrisburg.
3.—Each Engineering Division divided into three or four Engineering Districts (fifteen Engineering Districts in the State); each Engineering District in charge of a Resident District Engineer reporting to his Division Engineer.
4.—Each Engineering District divided into three or four Maintenance Districts (fifty-two Maintenance Districts in the State); each Maintenance District in charge of a Superintendent reporting

to his District Engineer. The Superintendent is in direct charge of operations. The snow-removal program mileage is divided into sections, each designated by a letter, beginning with "A" in each Maintenance District, and equipment assigned to care for each section.

A snow chart, kept at Harrisburg (like the chart described by Mr. Hamlin posted from wired reports from the superintendents to show any road that may be temporarily blocked or open only half width) is used for disseminating information to newspapers and for administrative control, but in each of the fifty-two districts similar control is kept by the superintendent, who remains at his post of control during the critical period of snow-removal operation. The maintenance superintendents have assistants who follow up the equipment and crews on the roads seeing to the actual details of the work and taking care of repairs or any emergencies that may arise. They and the equipment operators report to the superintendent by telephone from time to time as they reach certain objectives or require advice or assistance.

The Pennsylvania system for the distribution of equipment differs somewhat from the one described by Mr. Hamlin in that the tractors and trucks are not assigned to various foremen who are in charge of the maintenance of certain sections of road, but operate from one or more centrally located points (State garages or storage sheds) in each maintenance district; and the equipment operators report directly to the superintendents. The trucks and the tractors are charged with the responsibility of maintaining two-lane traffic ways. The foremen and caretakers in charge of the various sections are not all on regular snow duty, although some of them serve as equipment operators or helpers, but they are subject to call for emergency work and after each storm they are required to clean up the roadway, and open culverts, and side and shoulder drains.

The superintendents' control of snow removal in Pennsylvania is facilitated and assisted at need by co-ordination of equipment and supplies under the direction of the district engineers, and, in case of further need, the district engineers' equipment and supplies are co-ordinated by the division engineers.

One of the most valuable aids that Pennsylvania has experienced in handling its snow problem is the co-operation of the Weather Bureau stations, from which each superintendent secures advance notice of coming storms, which enables each outfit to be in readiness for duty when the time comes.

Location.—In a number of cases, it has been possible to diminish the snow-removal work by giving particular attention to snow in the location and design of the improvements. By relocation of certain sections, laying the road on wind-swept ridges rather than against hillsides; in other cases by raising the grade line in cuts; and in places where it has not been possible to relocate or raise the grade by widening cuts and flaring slopes, conditions have been improved. All over the State, but particularly in the sections that are most troubled with drifting, a point has been made to remove, or have removed, unnecessary obstructions that would create drifting tendencies.

Snow Fence.—For a number of years Pennsylvania has recognized the value of wind-breaks to prevent drifting. The ideal way to protect a submerged piece of road would be by a tall dense hedge. Such wind-breaks may be practicable in the future, but probably are not so at present. Substitutes can best be furnished by portable snow fence.

At present, the most economical fence available is the wire and picket type. Standards and specifications should be drawn with a view to permitting open competition and encouraging improvement as well as describing minimum requirements. It would be difficult to show just how much money is saved by the use of snow fence, but the writer is confident that it is more economical to prevent drifts than to open them.

Equipment.—The problem of equipment is primarily in the choice between specially built snow-removal machines and tractors or trucks with adjustable plows. The trucks and tractors used for snow removal are busy during the construction season on maintenance operations.

The tractor rotary is typical of the special machinery. The adjustable plow is either the V-type or mold-board, although recently the V-type has been improved and is generally in greater favor.

Expediency and economy are involved in the selection of equipment. A piece of equipment assigned to a section of road should be able to cope with all conditions on that section. All equipment should be charged to the work at sufficient rental to cover general charges, depreciation, interest, insurance, storage, etc., as well as actual operating costs; and if two or more types of equipment are capable of handling the work, that type which does the work at least cost, rental included, should be selected. Under some conditions special snow-removal equipment is desirable. For instance, the Pennsylvania Highway Department furnished rotary plows to the maintenance district that had the 137 in. of snowfall in the winter of 1925-26 and is satisfied that the greater cost was justified in keeping open roads that would have been blocked in spite of the efforts of less effective equipment. However, in other parts of the State where the fall was lighter, it was possible to keep the roads open and save money by using tractors and trucks fitted with adjustable plows.

It is not desirable to provide one piece of equipment for handling heavy drifts and other equipment for the light work between drifts. The amount of equipment provided is determined by the number of miles of road to be kept open and the number of miles each piece can handle. As Mr. Hamlin points out, there will be some variation in the length of the sections, but usually each piece of equipment will care for about 13 miles of road.

Operations.—Practical consideration sets a minimum limit of depth to about 2 in. for snow removal. Traffic is not seriously handicapped by less than a 2-in. depth and setting the plows closer to the pavement than this would increase the difficulty of operating. Accordingly, work customarily starts when the 2-in. depth is reached and continues as long as there is fall or drifting.

The roads should be kept open for two-way traffic if possible, but if storm conditions prevent this, a single track should be maintained open, with turnouts at intervals for passing. At the expiration of the storm, this track can be widened.

When the storm has subsided and the roadways of the program mileage are open to two-way travel, the time has come to proceed more cautiously. Such equipment as betrays need of repairs should receive immediate attention. Some may be required to shove back snowbanks from the shoulders, to make room to dispose of additional snow in case of another storm. The other equipment may be scattered to open drifts on roads not on the program; and the caretakers and foremen, or patrolmen, on the program mileage, should proceed under supervision to open culverts and to cut shoulder drains through the snowbanks along the pavement. This clean-up work is essential if the successive snow removal operations are not to be impeded by the accumulation of preceding storms and if, when the thaws come, the snow water is to be removed without damage to the road.

Personnel.—The personnel feature of snow removal appears to be not a problem but rather the solution of a problem. One of the most serious difficulties in road maintenance has been rebuilding the organization in the field after winter suspension of activities. Snow-removal work furnishes winter employment, and thus holds a number of the best maintenance men.

Made up of picked men from maintenance forces, the snow crews exhibit a high morale. Comparatively new as the snow-removal activity is, they have firmly established for it the tradition of efficient and unflinching service.

Costs.—Costs will naturally vary with conditions. The method of accounting used, whether depreciation and interest on equipment are included, for instance, will also affect the figures. In making any comparison, these things must be taken into account.

TABLE 2.—Analysis of Snow Removal Costs in Pennsylvania for the Winter of 1925-26.

Items.	Program only.
Snowfall (U. S. Weather Bureau report), in inches*	50.4
Snow fence,† in miles	101.0
Road,‡ in miles	5 061
Product of miles and snowfall, in inch-miles	225 074
Cost of cinders and erecting, dismantling, and storage of snow fence per mile of road	$15.95
Reported snow removal,§ in inch-miles	357 138
Percentage of excess of reported snow removal (due to drifting), in inch-miles	40
Cost of snow removal per mile of road	$109.09
Cost of snow removal per mile of road per inch of snowfall	$2.16
Cost of snow removal per inch-mile of reported snow removal	$1.55

* State average.

† Approximately 91 miles of picket and wire, and 10 miles of "railroad" types.

‡ Mileage on program kept continuously open.

§ An inch-mile is the quantity of snow of 1-in. depth, 24-ft. width, and 1-mile length.

An analysis of snow costs in Pennsylvania for the winter of 1925-26 (Table 2) is given with sufficient notes so that possibly parts of the data can be used for comparison in cases where the totals are not on equal basis.

George E. Hamlin,* M. Am. Soc. C. E. (by letter).—The value to the motor-vehicle operator and owner, of the free use of an unobstructed highway at all times, may be computed partly in terms of car-mile operation, including

* Supt. of Repairs, Connecticut State Highway Dept., Hartford, Conn.

interest and depreciation costs. This factor, however, is probably of small account considering the actual loss to the community in delayed delivery of needed items due to non-operation of the highway. The truck in many sections has taken the place of other types of transportation and a return to the original type cannot be made without preparation. Food supplies are not held in quantity in the retail store, dependence for replenishing the depleted stock being placed on the daily delivery by truck from the central warehouse.

Any business organization considers the minimizing of necessary losses as well as the making of profits. This is as true of highway operation as of any other business organization. Capital invested cannot economically be allowed to lie idle when a return can be furnished to the advantage of both the producer and the consumer.

The State Highway System of Connecticut, including bridges and ferries, had an estimated value in 1926 of more than $38 000 000. If interest at 4% only is considered, the daily charge will amount to $4 165, this being a total loss through non-operation of the highway system. The fees received from the gasoline tax, which in Connecticut are devoted in their entirety to maintenance and reconstruction of highways, are also lost. These fees during the fiscal year ending July 1, 1926, amounted to about $2 500 000 and in the three winter months of December, January, and February, to about $550 000, or about $6 100 per day. These two items of interest and loss of gasoline revenue alone amount to more than $10 000 per day.

The snow removal cost in Connecticut, as elsewhere, varies with weather conditions, but probably can be averaged at about $100 000 per year. This amount can be wholly compensated for by interest and loss of gasoline revenue charges for a 10-day period during the winter months.

An additional benefit from snow removal which is of measurable saving in maintenance costs is the reduction of necessary spring patching of surfaces. In this one item alone, the State of Connecticut has probably justified the entire cost of snow removal. Where snow is permitted to remain, ruts are formed in which practically all the travel operates. Each vehicle is necessarily equipped with chains, as icy conditions are continuously met. The travel is concentrated in narrow widths in which the chains cut and wear through the road surface for an appreciable depth. Water enters the roadbed through the broken surface and disintegration increases proportionately. This condition results in the necessity for continuous surface patching in the early spring.

Where the snow is removed, travel utilizes the entire width of the highway, the wear is more evenly distributed, and chains are the exception rather than the rule. The ordinary yearly bituminous application to the surface will place the roadway in good condition for summer travel, and patching—always the expensive treatment—is reduced to a minimum.

Maintenance can possibly be defined as the labor and material required to keep all highways at all times of the year in an operating condition commensurate with the just demands of travel.

The writer agrees with Mr. Means that snow removal costs should be a part of the annual maintenance budget.

Mr. Sevison has commented on the importance of proper location in construction so that the necessity of snow removal may be reduced to a minimum. In Connecticut, since about 1924, this has been one of the duties of the locating engineer and is definitely considered in determining the economic location of a new or relocated highway. When necessary, cuts are widened to reduce snow hazards and to permit the clearing to the sides of the roadway by plows, rather than to require removal by expensive hand methods.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1624

FINISHING AND CURING OF CONCRETE ROADS*

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With Discussion by Messrs. E. S. Borgquist, Herbert J. Gilkey, and C. L. McKesson.

Synopsis

Separate sections are devoted to the finishing and to the curing of concrete roads.

Finishing Concrete.—Modern practice is reviewed, with a description of acceptable appliances.

Data are given regarding tests for determining the relative efficiency of finishing machines of two well known types.

The use of the Vialog is recommended as an aid in securing a smooth pavement surface.

Curing Concrete.—The results of recent investigations are set forth in detail, including considerable research data not heretofore published.

The investigations described indicate that the period of watering may be much reduced from present standard practice. The expense of long continued watering constitutes an important item of cost in the construction of concrete roads, and any substantial reduction in the watering period, therefore, would result in an appreciable saving.

Experiments indicate that the average time during which the pavement is kept closed to traffic may be reduced. This reduction in the curing period would result in a lower cost for maintaining detours and in an even greater advantage to the motoring public through the reduced cost of the operation of vehicles over rough and sometimes lengthy detours.

* Presented at the meeting of the Highway Division, Salt Lake City, Utah, July 9, 1925.
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General

In 1925, the United States Bureau of Public Roads fixed the total mileage of concrete pavements in the United States at approximately 34 000, and each year many miles are being added to this total.

During the period of development of this type of construction, highway engineers have been fully alive to their responsibility in the expenditure of the huge sums of public funds for this work, and many separate agencies comprising the engineering bureaus of Governmental departments, State highway organizations, municipal engineers, colleges, and universities, various technical societies, the Portland Cement Association, and a host of private investigators, have been diligently studying the problems involved in this popular type of construction.

Many of the problems of design and construction have been solved and specification requirements are now quite well standardized, including the use of a rich concrete (1 : 1.5 : 3 to 1 : 2 : 4) in a one-course pavement. Thickened edges and longitudinal center joints have been adopted by many of the States. The time of mixing has been fixed at 1 min. by thirty-four States, the others requiring 1¼ to 1½ min. The grading of aggregate has not been standardized and perhaps never will be, owing to widely differing characteristics of materials in various sections.

The finishing of concrete pavements has been the subject of intensive study by all highway engineers because roughness is so easily discerned both by engineers and by the traveling public. News of innovations in finishing methods and equipment has traveled fast, and as a result methods of finishing and appliances have become fairly well standardized.

Finishing Concrete

Hand-Finishing.—A heavy hand template or strike-board is required by nearly every State. The design of this template is usually specified, likewise the weight as from 11 to 20 lb. per ft. of length. A weight of 15 lb. per lin. ft. is adequate and is not too heavy for manipulation. Such a template should be rigid in construction and made so that it can be adjusted to eliminate sag and to provide the desired crown in the pavement.

A straddle-float for finishing at joints is required by thirty-two States. The latest and most efficient type has a metal frame and renewable or metal-protected boards on the finishing face. Such a float should be about 6 ft. long and approximately 5 in. wide with a handle long enough to permit its operation from the side of the pavement.

Rollers are required by many States. These rollers, 8 to 10 in. in diameter and 4 to 6 ft. long, have been considered useful in removing the surplus water which collects on the surface of the concrete after tamping and floating. The California Highway Commission, by decreasing the quantity of mixing water to the limit consistent with proper finishing, has largely eliminated this surplus water so that the roller is no longer regarded as essential.

Belts have been widely used and have given good results. A ¾-in. hose used in the same manner as a belt has been found satisfactory on some jobs.

A flexible finishing board about 12 in. wide, with a length equalling the width of the pavement, and provided with "plow" handles, is used in some States and is believed by them to excel the belt in that it does not cup and sag and in that its movements can be controlled more accurately. It is the writer's belief that the finishing board used transversely tends to preserve the crown better and that it gives a smoother surface than a belt. This same type of finishing board gives excellent results when operated longitudinally instead of transversely, the operators working from bridges. When used longitudinally, the equipment is usually known as the "longitudinal float" and in some cases it is made rigid by the addition of a 2 by 4 or 2 by 6-in. piece nailed to the upper surface.

Straight-edges from 6 to 10 ft. in length are necessary on every project if satisfactory finishing is to be secured. A split straight-edge 6 ft. long is a valuable instrument for checking up joints. The handle should be light and of sufficient length to enable the user to test an entire joint from one side of the pavement. Such straight-edges, made from aluminum, are now available. They have the merit of lightness and freedom from warping. The State of Washington used such a straight-edge with a short section of projecting "fin" or knife-edge. Whenever the straight-edge is placed on the fresh concrete, the "fin" leaves a slight but unmistakable mark on the pavement and thus creates a permanent record of its use. Inspectors are instructed to test each joint carefully, and the presence of these marks on the pavement furnishes indisputable evidence as to their compliance with instructions. A satisfactory finish is the result of the use of proper appliances by skilled men, and specifications should clearly describe both methods and equipment.

Well-mixed concrete of uniform consistency is necessary for first-class work. Coarse sand and dryer consistencies of concrete somewhat increase the finishing difficulties, but complaints from the finishers should not be justification for the use of fine sand or sloppy concrete. A vast amount of research work has shown that fine sand or a wet mix results in inferior strengths and wearing qualities in a pavement. The California Highway Commission has adopted the truncated cone slump test for controlling consistency and specifies coarse sand. A maximum slump of 1 in. for machine finishing has been found practicable. These requirements have not resulted in added cost and they insure a concrete of compressive strength of 3 000 to 5 000 lb. in 28 days and with a satisfactorily smooth surface as evidenced by Vialog readings.

Machine Finishing.—Some of the foregoing discussion of equipment and methods applies equally well to machine finishing. Nine States now require machine finishing; in eleven it is optional with the contractor. One State is reported as prohibiting machine finishing and a number of States regulate the use by special provisions. In mountainous regions some difficulty is experienced in the use of mechanical finishing equipment due to the frequent necessity for widening pavements, on curves and the use of a flat cross-section on superelevated curves. Frequent expansion joints somewhat hamper the operation of mechanical finishers, but on sections having long tangents mechanical finishers are efficient and economical.

In California the question has been raised as to the relative efficiency of the so-called "tamping machines" as compared with "finishing" or "screed" machines not provided with a tamping strike-board. The latter type depends on the transverse movement of the finishing board and the compacting action of the surplus material carried ahead of the blade that serves as a strike-board. The California Highway Commission made a somewhat limited study to ascertain, if possible, the relative efficiency of these two types of equipment and of hand-finishing.

Sections selected on a paving project near Sacramento were finished with the "tamper" type, with the finishing machines, by hand, and one section by the tamper and finisher used in succession.

Cores were drilled from these sections and the voids in the samples determined. The cores were then tested for compressive strength and finally Vialog readings were taken to determine the relative roughness of the various sections. The average weight per cubic foot of the concrete was compared with the weight of concrete in 6 by 12-in. cylinders, made according to the standard requirements of the American Society for Testing Materials at the time the concrete was being laid. Table 1 gives the results.

TABLE 1.—COMPARISON OF TESTS ON CORES FINISHED BY DIFFERENT METHODS.

Method of finishing.	Cores.			Weight, 6 by 12-in. cylinder from same station, in pounds per cubic foot.	Relative density, assuming cylinder = 100%.	Average compressive strength of cores, in pounds per square inch.	Compressive strength, cylinder from same station, in pounds per square foot.	Strength ratio: cores to test cylinders, percentage.	Vialog record, inches per mile.
	Percentage of voids.	Specific gravity.	Weight per cubic foot.						
Tamping machine	8.82	2.29	142.45	148	96.25	3 645	2 578	142	20.3
Hand-tamped and finished.	9.72	2.28	142.09	146	97.32	3 303	2 593	128	....
Tamping machine and finishing machine	10.81	2.26	140.99	147	95.91	3 347	2 593	129	....
Finishing machine	10.88	2.24	139.24	146	95.37	3 840	2 640	145	21.7

In these tests the tamping type machine had a slight advantage as to the density and smoothness of pavement, and the finishing machine had a trifling advantage as to strength ratio. The showing was considered as somewhat favoring the tamping type but the results did not justify the exclusion of the non-tamping type of finisher.

Relative Smoothness Determinations.—The Vialog has been used by the California Highway Commission as a means for determining the relative smoothnesses of various sections of pavement. This is an instrument attached to an automobile to determine the degree of roughness of the road by recording the vertical motion of the automobile body with reference to its front axle. The utmost care is necessary if the results are to have any real significance. The condition and inflation of tires must be uniform at all times.

Standardization tests must be made at frequent intervals and skill and uniformity in the operation of the car are essential.

The Vialog, however, provides the only means available for measuring surface irregularities over long stretches of pavement, and in spite of its defects its use has been found to be well worth while.

Vialog readings taken at frequent intervals during construction, stimulate the field forces in their efforts to secure a smooth pavement. Some check is also possible as to the relative efficiency of various finishing methods and equipment. During the 1924 construction season, the Vialog readings indicated surface roughnesses on concrete pavements varying from 8.9 to 30.5 in. per mile. Vialog readings taken during the 1925 construction showed a substantial decrease in roughness, in one instance to as low as 6 in. per mile, with the Vialog operated at the standard speed of 20 miles per hour.

Curing of Concrete

Present Practice.—As has been pointed out the practice with reference to the finishing of concrete has been more or less standardized throughout the United States; but the same cannot be said of curing.

All the State Highway Departments specify covering with canvas or burlap immediately after placing and subsequent sprinkling, to prevent hair-cracks until the pavement is ready for covering with earth or for ponding. In many States, covering with wet earth and ponding are considered as alternatives optional with the contractor, but in some of the States either one or the other method is definitely specified. Calcium chloride curing is now permitted in several of the States.

The wide variations in curing practice are in the length of time which the pavement must be kept wet, the number of days during which it must be kept covered, and the number of days which must elapse before it is opened to traffic. The time of watering varies from 8 to 18 days and the time of opening to traffic, from 13 to 30 days.

Long-continued watering of concrete involves an important item in the cost of construction. Keeping the pavement closed to traffic during the curing period involves the expense of maintenance of detours and quite often inconvenience and increased operating expense to automobile owners. It will be agreed that the watering and curing periods should be reduced to the minimum consistent with safe practice.

Recent Curing Studies.—Since 1920, several elaborate studies have been made for the purpose of determining the relative efficiency of various methods of curing. The State Highway Department of Illinois experimented quite extensively with the use of calcium chloride and other deliquescent salts, and a detailed report covering their investigations was presented by H. F. Clemmer, M. Am. Soc. C. E., and Mr. Fred Burggraf* in 1923.

In 1923, the Structural Materials Research Laboratory of Lewis Institute, Chicago, Ill., conducted a series of curing tests. The California Highway

* *Proceedings*, Am. Soc. for Testing Materials, Vol. 23, Pt. 2, 1923.

Commission, in co-operation with this Laboratory, likewise conducted an extensive series of curing tests at Sacramento, in 1924.*

The California Highway Commission, in 1924, conducted an independent series of field investigations for the determination of the relative efficiency of wet earth curing and calcium chloride curing.† With the results of the previous tests at hand and with the desire of definitely justifying certain tentative conclusions previously reached, the California Highway Commission during 1925 conducted an additional series of experiments.

As the data from all these experiments will be referred to in the following discussion of the curing methods, it seems desirable to include a brief description of the scope of each of these studies.

The Illinois series included tests of more than 500 concrete slabs, 8 by 12 by 30 in., subjected to various methods of curing. The co-operative series conducted by the California Highway Commission and the Structural Materials Research Laboratory included flexural strength tests of more than 500 beams, 7 by 10 by 38 in., cured with an earth covering watered for varying periods, with curing paper, and with calcium chloride. This co-operative investigation will, hereinafter, be referred to as the Sacramento Tests. The field tests, series of 1925, by the California Highway Commission, include 105 cores drilled from pavement cured by covering with earth and watering 0, 3, 7, 8, 10, 12, and 14 days and tested at 14, 21, and 28 days. Sixty-four reference specimens consisting of cylinders, 6 by 12 in., were also made from the same concrete in connection with this series and cured in the laboratory in water for 28 days.

Water Curing.—To analyze and correlate the results of the various investigations, some consideration should first be given to the objective to be attained. This objective consists in the provision of sufficient water to carry on hydration during a stated curing period. It is well known that hydration is rapid in the earlier stages of curing and that it proceeds more slowly after the first few weeks. It has been shown by W. K. Hatt,‡ M. Am. Soc. C. E., and by others, that the presence in concrete of free moisture results in a reduction in strength. It would seem reasonable to assume that the most efficient method of curing would be that which would provide sufficient water for hydration during a given curing period without a surplus remaining as free moisture at the time of testing.

In the research investigations in which the California Highway Commission has been interested, special attention has been given to the moisture content in the concrete at various stages of curing, and it is believed that the assumption just stated has been verified in principle. If the assumption is correct that the quantity of water necessary for hydration varies with the length of the curing period, and that the presence at the time of test of an excess of water in the concrete results in lowered strengths, it must follow

* The results of this series have been covered in a report published as *Bulletin No. 15*, by the Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

† "The Curing of Concrete with Calcium Chloride," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *California Highways*, Vol. 2, p. 5, February, 1925; *Public Roads*, Vol. 6, No. 1, March, 1925.

‡ "Effect of Moisture on Concrete," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 270.

that the period of watering should vary with the length of the curing period. All the tests herein discussed indicate that this is true.

The length of the curing period is the time necessary for concrete to develop sufficient strength to permit of its safe use for the purpose for which it was constructed. In the design of concrete for structures, the ultimate compressive strength is frequently assumed to be 2 200 to 2 500 lb. per sq. in. In pavement, stresses and loadings are much more uncertain, but concrete having a compressive strength of 2 000 to 3 000 lb. per sq. in. is usually considered safe for use. The necessary curing period for concrete in pavements or in structures is, therefore, a period of time which, with suitable curing, will enable it to develop these strengths.

Concrete made from satisfactory materials continues to develop strength for years after the termination of the curing period. This increasing strength adds to the factor of safety. It is not necessary, and it may not be possible, to supply water during a curing period in sufficient quantity to carry on hydration for a period of years.

It has been shown* that if hydration in concrete is stopped due to a lack of moisture, it will resume when water is applied even after a long lapse of time, and that thereafter the concrete will increase in strength more or less normally. Concrete in roads will be subjected to occasional rainfall after the curing period ends and moisture for continued hydration will thus become available from time to time. In the curing of concrete roads, therefore, the engineer is most concerned in supplying the water necessary to develop the required strength in the earliest possible time.

Quantity of Water for Curing.—In the Sacramento Tests, concrete of normal consistency contained mixing water amounting to about 8% of the combined weight of the cement, sand, and gravel. It was found that the original mixing water, if conserved, was sufficient to carry hydration through a 90-day curing period. The relative strengths obtained by some of the more important methods of curing are shown in Fig. 1, which also shows the change in moisture content in the concrete measured in terms of original mixing water. It will be seen that concrete which had no watering lost about 32% of its moisture while concrete watered only three days, lost only 7.1% of the original mixing water. This indicates that treatment during the first three days after the placing of concrete has much to do with subsequent strengths.

The rate of loss of moisture is dependent on a number of conditions, including the size and surface area of the specimen, the temperature and humidity of the air, and the condition of the sub-grade. In the Sacramento Tests, specimens were cured on water-proof paper to prevent the escape of water to the sub-grade, and a careful record was made of the humidity and temperature. The temperature was high and the humidity low during these tests—conditions that were doubtless more favorable to rapid drying than those of the Illinois tests, probably as severe in fact as may ever be encountered.

* "Effect of Age and Condition of Storage on Compressive Strength of Concrete," by H. F Gonnerman, *Proceedings*, Am. Concrete Inst., 1918.

The results of the several tests herein described are applicable to pavements or other concrete resting on a sub-grade and exposed on one surface only. They would also probably apply to concrete in walls or abutments, but concrete in thin members in structures has a much larger relative surface area and probably a more rapid evaporation.

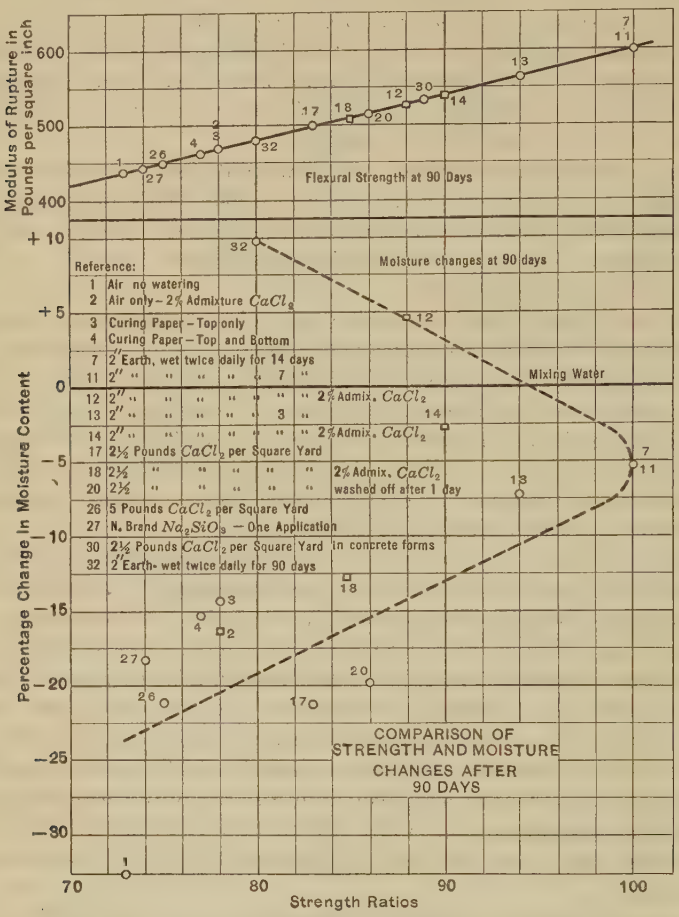
Modulus of Rupture in Pounds per square inch
600
500
400
Flexural Strength at 90 Days
+10
+5
0
−5
−10
−15
−20
−25
−30
Percentage Change in Moisture Content
Moisture changes at 90 days
Mixing Water
Reference:
1 Air no watering
2 Air only – 2% Admixture $CaCl_2$
3 Curing Paper – Top only
4 Curing Paper – Top and Bottom
7 2" Earth, wet twice daily for 14 days
11 2" " " " " " 7 "
12 2" " " " " " " 2% Admix. $CaCl_2$
13 2" " " " " " 3 "
14 2" " " " " " " 2% Admix. $CaCl_2$
17 2½ Pounds $CaCl_2$ per Square Yard
18 2½ " " " " " 2% Admix. $CaCl_2$
20 2½ " " " " " washed off after 1 day
26 5 Pounds $CaCl_2$ per Square Yard
27 N. Brand Na_2SiO_3 – One Application
30 2½ Pounds $CaCl_2$ per Square Yard in concrete forms
32 2" Earth wet twice daily for 90 days
COMPARISON OF STRENGTH AND MOISTURE CHANGES AFTER 90 DAYS
70
80
90
100
Strength Ratios

FIG. 1.

In connection with the curing tests of concrete beams, a study was made of the loss of water in concrete placed in pans and exposed in sunshine. A similar test was included in the Illinois series. The results of both tests are shown in Fig. 2.

In both tests, the loss of moisture was rapid during the first three days of exposure. After that, it was much less. As would be expected, the early loss was greater in Sacramento than in Illinois.

In Fig. 1, moisture changes in the concrete during the 90-day curing period are expressed as percentages of original (mixing) water content. It

will be seen that watering up to the time of test (90 days) resulted in an increase of moisture over the quantity used in mixing (see Item 32). This method of curing provided free moisture not necessary for hydration and appreciably decreased the strength. These tests indicate that at 90 days there is a well-defined relation between moisture content and flexural strength in concrete. A comparison of the upper and lower curves in Fig. 1 indicates that a substantial increase or decrease in water content (based on mixing water) is reflected by a decrease in strength.

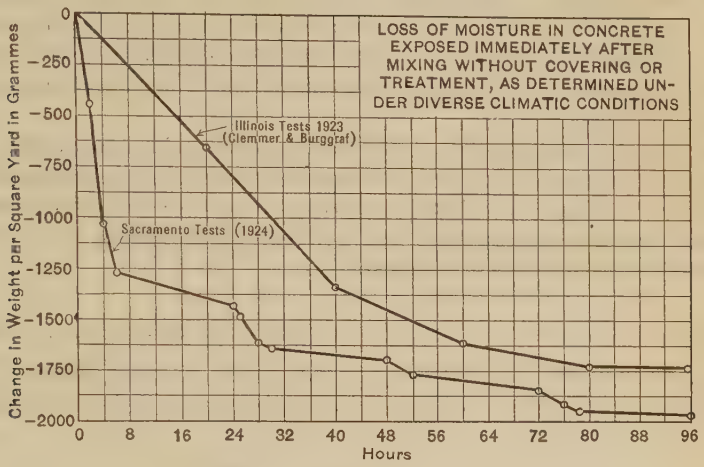

FIG. 2.

Influence of Temperature on Curing.—It has long been known that the early hardening of cement is retarded by low temperatures and accelerated by high temperatures. Variations in temperature during the curing period similarly affect concrete.

In considering the relative efficiency of various curing methods where early strengths are required it is reasonable, other conditions being equal, that preference should be given to a method which would provide the highest temperature in the concrete during the curing period. Ponding or covering with earth, wet twice daily, results in a considerably lower temperature in the concrete than exposure under a dry earth covering or without any covering.

A short watering period permits of the earlier drying of the covering and of increased temperature thereafter. It seems reasonable that this had its influence in producing relatively high strengths in concrete cured with little or no watering, as described in the following test results.

All the California curing tests were conducted in summer and should not be applied without adjustment to winter conditions. It is quite possible that the time of opening might be based on the mean temperature during the curing period.

Relative Efficiency of Various Wet Curing Periods.—The 1925 series of field curing tests by the California Highway Commission was made on a pavement-widening project fourteen miles from Sacramento. The strip of pave-

ment was 6 ft. wide and 7 in. thick. The concrete was mixed in a central mixing plant and hauled to the work in side-dump trucks. All the mixing, placing, and finishing were conducted in the usual way. Seven separate sections, each representing a day's run of from 500 to 1 500 lin. ft., were designated for the test and each section received different curing treatment. All the sections were covered with burlap immediately after laying and with earth on the following morning. Section 1 was watered 14 days; Section 2, 12 days; Section 3, 10 days; Section 4, 8 days; Section 5, 7 days; Section 6, 3 days; and Section 7, 0 days. In all cases the earth covering was removed in 14 days. At the time of the placing of concrete, 6 by 12-in. cylinders in pairs were taken at five stations in each section. The 10 specimens, representing each section, were cured in water in the laboratory for 27 days and broken on the twenty-eighth day.

One core was drilled at each of the five stations in each section for 14, 21, and 28-day tests. Each age and condition was thus represented in the test by five cores. Some honeycombed or otherwise defective cores were obtained, but these were rejected and drilling was continued until a full set of apparently perfect cores was secured.

TABLE 2.—SACRAMENTO CURING TESTS (1925) FOR COMPRESSIVE STRENGTHS.

(6 by 12-Inch Moulded Cylinders and 4½ by 7-Inch Drilled Cores. Concrete Mix, 1 : 1.9 : 3.8, Using 6 Sacks of Cement per Cubic Yard.)

Number.	Watering period, in days.	Strengths.						
		14 Days.			21 Days.		28 Days.	
		* 28 days, 6 by 12-in. cylinder.	† Cores, in pounds per square inch.	‡ Strength ratio.	† Cores, in pounds per square inch.	‡ Strength ratio.	† Cores, in pounds per square inch.	‡ Strength ratio.
Section 1	14	3 296	3 285	100	3 873	118	4 229	128
" 2	12	3 105	3 468	112	4 103	132	4 414	142
" 3	10	2 930	3 123	107	3 694	126	4 225	144
" 4	8	2 968	3 366	113	3 483	117	3 877	130
" 5	7	3 118	3 073	99	3 596	115	4 101	132
" 6	3	3 356	3 948	118	4 323	129	4 068	121
" 7	0	3 378	3 329	99	3 436	102	3 814	113
Averages	..	3 164	3 370	106	3 787	119	4 104	130

* 6 by 12-in. cylinders made in accordance with A. S. T. M. standards, and cured in water for 28 days. Each strength is an average of ten specimens, except Section 7 in which only four specimens were made.

† Cores drilled with Calyx shot drill. The cores tested were free from visible defects. Each strength is an average of five cores.

‡ Strength ratios are ratios of core strengths to 28-day strength of cylinders from the same section.

Table 2 gives the results of the tests of these cores and Fig. 3 indicates the relative efficiency of various periods of watering. Curing without water gave high strength ratios in the early tests showing that mixing water, con-

served to some extent by a covering of dry loose earth, provided nearly enough water to insure proper hydration through the early curing periods. In drilling, it was found that the surface of the concrete cured by this method was noticeably softer than that from sections which had received water treatment. This also verifies the observations* of D. A. Abrams, M. Am. Soc. C. E., and the results of the Sacramento curing tests in which it was found that concrete which received no water had only about three-fourths of the surface hardness of similar concrete cured in water even for 3 days.

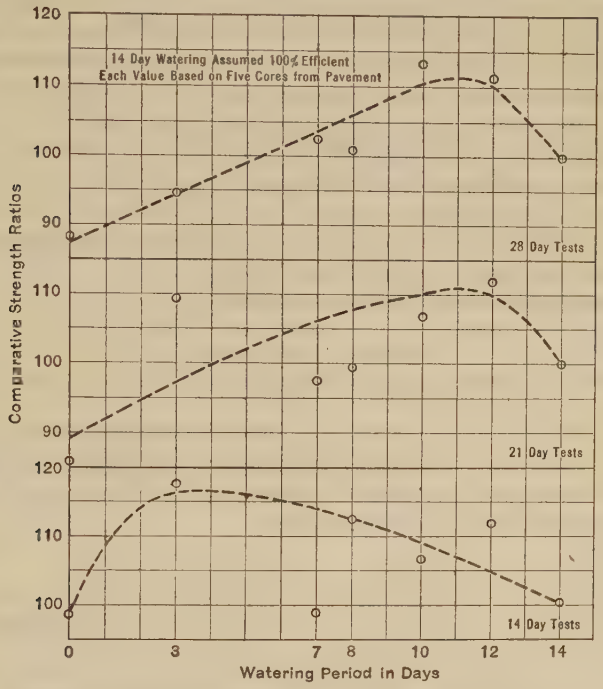

FIG. 3.—SACRAMENTO TESTS, 1925, RELATIVE STRENGTH RATIOS FOR VARIOUS PERIODS OF WATERING.

Watering for 3 days gave the highest strength in the 14-day tests. This verifies some of the Illinois tests and almost exactly checks the Sacramento tests of 1924. It is quite apparent that 3-day watering provides all the moisture necessary to carry on hydration through a 14-day, or even a 21-day, curing period. At the same time, it permits of a higher temperature in the concrete at an early period in the curing and thus accelerates early hardening. It does not provide any free moisture to reduce the strength at the time of the test.

Watering for 7 and 8 days gave satisfactory strengths at all ages. The strengths and strength ratios for these two treatments were about the same except for the 14-day tests. In this average, one of the five cores broke at an unaccountably low stress, thus reducing the average somewhat.

* "Wear Tests of Concrete," by D. A. Abrams, *Bulletin 10,* Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

Watering for 10 and 12 days gave about the same strengths as the shorter watering periods in 14-day tests, but showed a large gain in the 21 and 28-day tests.

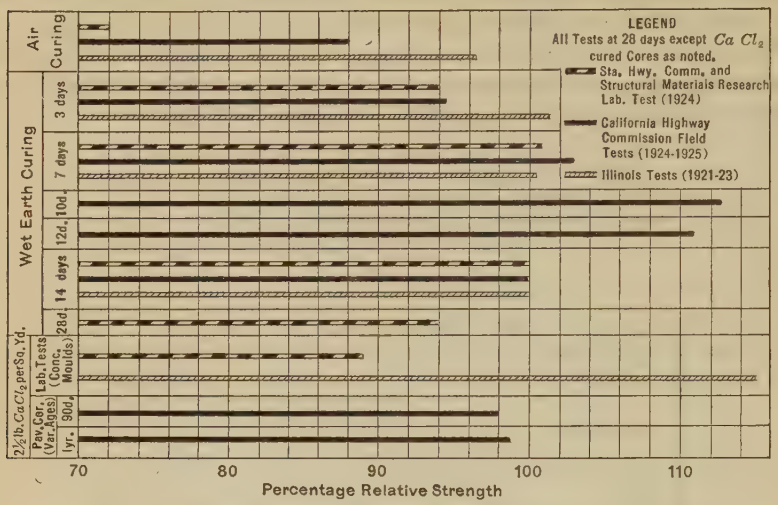

FIG. 4.—CONCRETE STRENGTHS FOR VARIOUS TYPES OF CURING.

Watering for 14 days, heretofore accepted as the standard in California, fell behind other periods of watering as to strength in almost every instance. It made a little better showing than curing without water, but was decidedly superior in surface hardness.

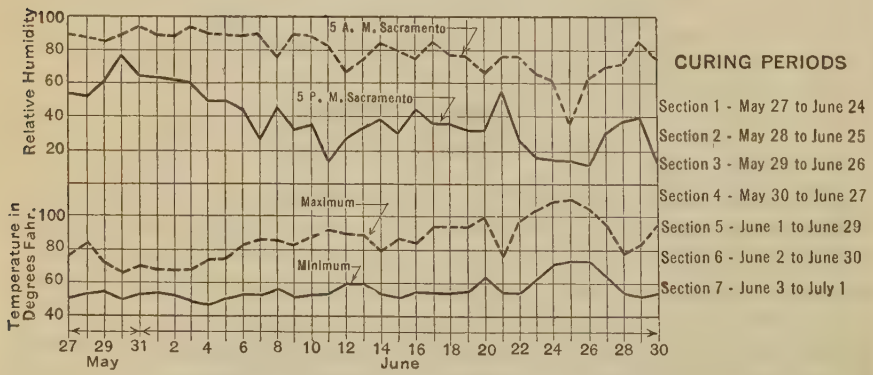
Relative Humidity
100
80
60
40
20
5 A. M. Sacramento
5 P. M. Sacramento
Temperature in Degrees Fahr.
Maximum
Minimum
27 29 31 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
May
June
CURING PERIODS
Section 1 - May 27 to June 24
Section 2 - May 28 to June 25
Section 3 - May 29 to June 26
Section 4 - May 30 to June 27
Section 5 - June 1 to June 29
Section 6 - June 2 to June 30
Section 7 - June 3 to July 1

FIG. 5.—VARIATION OF TEMPERATURE AND HUMIDITY DURING THE SACRAMENTO TESTS, 1925.

Fig. 4 shows a comparison of the results of this test with those obtained in the Sacramento tests of 1924. It also shows the results obtained in Series 1 of the Illinois tests.

Fig. 5 shows the variation of the temperature and humidity during the period covered by the field tests (1925).

It will be seen that the curing period included some very hot dry weather. There was no rainfall after the expiration of the watering periods and only 0.05 in. during the entire period of the test.

In Fig. 6, the results of the field tests (1925) are shown in comparison with the results obtained in the Sacramento Series (1924). It will be seen that the 1925 tests strongly corroborate the earlier tests. They show that about 90% of the 21-day strength is obtained in 14 days. It is worthy of note in this connection that only 6 of the 105 test cores fell below 3 000 lb. in strength and all the group averages are above this figure. If 3 000-lb. concrete may be considered safe for use, a 14-day curing period and 3 days or more of watering could safely be counted on to give the desired strength with first-class concrete.

Calcium Chloride Curing.—A study of the foregoing results and the knowledge that mixing water, if conserved, is sufficient to carry on hydration, even through a 90-day period, throws some light on the subject of the curing of concrete by the application of calcium chloride. Calcium chloride deliquesces readily when the atmosphere has a high relative humidity. At low humidities,

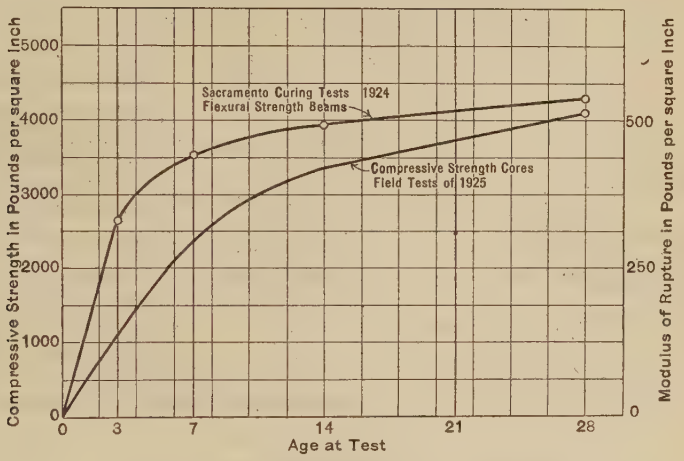

FIG. 6.—COMPARISON OF 1924 AND 1925 TESTS.

probably 40 or lower, it yields up a part of the water thus accumulated. Under favorable conditions it acts more or less as a reservoir and perhaps supplies some water to the concrete beneath. Under conditions of low relative humidity, it is equally likely to deliquesce by removing water from the concrete, which is needed to carry on hydration.

In 1924, a number of field tests were made by the California Highway Commission to determine the relative efficiency of 14-day water curing and curing by the application of 2½ lb. of flake calcium chloride per sq. yd. The results of these tests appear in Table 3, including cores drilled recently at an age of approximately one year. In these tests, calcium chloride was found to be 81 to 98% efficient in earlier ages. At one year, there is little or no difference in the strengths of cores cured with water or with calcium

chloride. During the winter months there was about 17 in. of rainfall and this supplied moisture for continued hydration.

In view of the fact that 3-day water curing is effective even under most adverse conditions, it does not seem that recourse to calcium chloride would be justified except under unusual circumstances, such as desert construction; and under such conditions, the tests do not indicate high efficiency for the calcium chloride.

TABLE 3.—COMPARISON OF CURING BY WATER AND BY CALCIUM CHLORIDE.

Location of experiment.	Number of samples.	Average Compressive Strength, in Pounds.				Efficiency of calcium chloride,* percentage.	Remarks.
		6 by 6 by 12-Inch Moulded Specimens.		4½-Inch Cores from Pavement.			
		Cured with water, 28 days.	Cured with calcium chloride, 28 days.	Cured with water.	Cured with calcium chloride.		
Sacto-Stockton Highway, near Sacto, Calif.	24	.....	.....	4 292	4 218	98	Hot and dry. Water-cured concrete tested at 57 days. Concrete cured with calcium chloride tested at 70 days.
	20	.....	.....	5 064	5 042	99	Cores drilled and tested at age of 1 year.
Ventura-Los Angeles Highway, near Calabasas, Calif.	32	3 064	2 783	.....	.....	91	Hot and dry.
	28	.....	.....	4 057	3 294	81	
Los Angeles Highway, near Newbury Park.	12	.....	.....	3 491	3 093	88	Hot and dry.
Eureka-Arcata Highway, at Arcata, Calif.	72	3 887	3 545	.....	.....	92	Experiment in October. Weather cool; humidity high (70 to 90 per cent.). Some rain.

* Efficiency of calcium chloride curing, assuming water curing to be 100% efficient.

CONCLUSIONS

These test data seem to justify the following conclusions:

1.—That 7 days of watering is ample to insure proper hydration in concrete through a curing period of 28 or 90 days.

2.—That this watering period might be reduced to 3 days if the concrete is of first-class quality, thereby effecting a substantial saving.

3.—That, during summer weather, pavement constructed of first-class concrete may safely be opened to traffic in 14 days. (During cold weather, the hardening of concrete is retarded and the time of opening should depend on the actual conditions in each case.)

ACKNOWLEDGMENT

Grateful acknowledgment is made of the assistance rendered to the writer in connection with the planning and conduct of the foregoing tests by R. M.

Morton, Assoc. M. Am. Soc. C. E., State Highway Engineer, T. E. Stanton, M. Am. Soc. C. E., Assistant State Highway Engineer, C. S. Pope, M. Am. Soc. C. E., Chief Construction Engineer, all of the California Highway Commission, and of the courtesy of the Structural Materials Research Laboratory of Chicago, Professor Abrams in charge, in permitting reference to the results of the co-operative tests made at Sacramento in 1924 by the California Highway Commission and the Structural Materials Research Laboratory prior to the publication of the report of such tests. H. F. Gonnerman, Associate Engineer, represented the Structural Materials Research Laboratory in the Sacramento Curing Tests in 1924 and in the preparation of the report of that series of tests. The writer takes pleasure in acknowledging his valuable assistance in connection with the curing tests and in the interpretation of the results. F. T. Maddocks, Testing Engineer, and W. B. Stonebraker, Assistant Testing Engineer, of the California Highway Commission, also rendered able assistance in connection with the tests and in the preparation of data for this paper.

DISCUSSION

E. S. Borgquist,* M. Am. Soc. C. E. (by letter).—Perhaps no concrete is subjected to more severe usage than a road slab. Besides straight compression and frequent flexure stresses across weak spots in the sub-grade, there are heavy impact stresses, together with a continual abrasive action of different kinds of tires. Necessarily it takes a high quality of material to resist these forces and for this reason highway builders have been eager to adopt the best methods available for mixing, placing, and curing of concrete.

The State of California has built a great many more miles of concrete than any other Western State, and it is impressive to note the magnitude of the tests and the care with which they are carried out. The State of Utah has built more than 200 miles of concrete roads and has tried to keep abreast of the best practice.

A statement of mix in terms of 1 : 2 : 3 or 1 : 2 : 4 is only relative. The Utah State Road Laboratory has proved that it is entirely practicable to design a mix after the method developed by D. A. Abrams, M. Am. Soc. C. E., by taking into account the grading of the materials used as expressed in his "fineness modulus".

An inspector of ordinary practical ability can readily be taught to screen gravel and sand and so compute the fineness modulus of each. The sand to be used is found by Professor Abrams' formula:

$$p = \frac{A - B}{A - C}\,(100)$$

in which,

P = the percentage of sand to be used in a given mix;
A = the fineness modulus of the gravel;
C = the fineness modulus of the sand; and
B = the fineness modulus of the mixed aggregate.†

In road work this is no longer a purely academic formula for laboratory processes but a thoroughly workable method for determining how much sand to use in the field. By means of this formula the writer has found that, varying the sand content in concrete according to the grading of the sand and gravel, the mix was just as workable with 23% of sand as with 50%, because the sand in the latter case was very coarse, in fact, almost a pea gravel.

In regard to water content, the State road engineers have found that they can work with a much dryer mix than they ever thought possible. If, as proved by Professor Abrams, a concrete more than twice as strong results from simply cutting down the percentage of water from 1.0 to 0.5, measured in volume of water as compared with volume of cement, surely it is worth trying to obtain the additional strength, particularly for road slab concrete. The 0.5 coefficient gives a concrete too dry for anything except laboratory test work, but it is possible to cut down the percentage of water to 0.7 and still obtain an entirely

* Asst. Prof. of Civ. Eng., Univ. of Arizona, Tucson, Ariz.

† *Bulletin No. 1*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

workable mix, if striking and finishing of the road slab is done with a finishing machine.

On a 10½-mile concrete job poured in 1922, on which the writer served as Resident Engineer, a 1:5 mix was used and a test block was made every day from the concrete dumped on the sub-grade, with the result that the whole season's work showed an average compressive strength of 4 300 lb. per sq. in. for 30-day concrete. Of course, only good clean gravel and washed sand were used.

Relative to expansion joints, Mr. McKesson states that it is California practice to place a joint every 30 or 40 ft., regardless. In Utah, the practice is to place expansion joints only at the end of the forenoon run and of the afternoon run, thus giving two joints to be placed each day. It is difficult to obtain a "perfect" joint. The finishers usually build the joint too high so that there is a definite jar to a car riding over it. After the concrete has cured, the road will crack transversely every 30 or 40 ft., giving a contraction joint which opens at night and closes again on hot days. The joints thus formed by "Nature" occur only as needed. They are at least smooth to ride over and are quite as easily maintained as the expansion joints regularly placed square across the road at intervals of 30 or 40 ft. In Utah, one ⅝-in. elastite joint filler is used for each 25 lin. ft. poured, with a maximum of four such fillers placed side by side, thus giving a joint 1½ in. wide.

In the matter of curing, the State favors the method of "diking". In the West it is a difficult matter to get a contractor to keep the concrete damp if it is simply covered with a blanket of earth. If dikes are built and water is placed at least 3 in. deep on the slab, the concrete is much more likely to be properly cured and if it gets dry that fact is easily detected. Only in exceptional cases of scarcity of diking material is the sprinkling method allowed.

Herbert J. Gilkey,* M. Am. Soc. C. E. (by letter).—During the period 1924-26, the writer conducted several carefully planned series of tests in an effort to obtain basic knowledge regarding the curing process in its relation to the weight and compressive strength of Portland cement mortars and concretes. He has, therefore, given close attention to the several recent reports† of the California and California-Lewis Institute tests, that form the basis for that part of Mr. McKesson's paper relating to the "Curing of Concrete" (page 141).

It would be difficult to conceive of a more auspicious setting for a series of tests. They were made at public expense and apparently, therefore, without serious handicap for lack of funds. They were conducted under experienced, and presumably expert, supervision and in conjunction with the actual work of one of the leading State Highway Departments of the United States.

* Associate Prof. of Civ. Eng., Univ. of Colorado, Boulder, Colo.

† "Studies of Curing Concrete in a Semi-Arid Climate," by H. F. Gonnerman and C. L. McKesson, Assoc. M. Am. Soc. C. E., *Bulletin No. 15*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., August, 1925; "Curing of Concrete, Recent California Experiments," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *California Highways*, January, 1926; "Curing of Concrete, Recent California Experiments," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *Western Construction News*, April 25, 1926; "Studies of Curing Concrete in a Semi-Arid Climate," by C. L. McKesson, Assoc. M. Am. Soc. C. E., *Engineering News-Record*, March 18, 1926; Discussion, *Proceedings*, Am. Concrete Inst., 1926, p. 432; *California Highways*, Vol. 2, February, 1925, p. 5; *Public Roads*, Vol. 6, March, 1925, No. 1.

In so far as the tests and numerical data are concerned it is doubtful if any agency could have turned out better controlled or more accurate work for the given conditions. This is a very valuable and timely paper because it collects as one target a group of current fallacies that urgently need correction. It is an excellent illustration of careful, costly work largely neutralized because of misinterpretation and incomplete analysis.

With the exception of that part of the discussion that pertains to Table 1, what is herein said applies only to the part of the paper devoted to the "Curing of Concrete". Moreover, no consideration will be given to the data on the use of curing compounds and admixtures. No direct notice will be taken of the flexural test results, although much of what will be said applies equally to them. In general, strong concrete is strong in each of the several ways by which strength may be judged. It is of the curing process in its relation to compressive strength that the writer feels competent to speak with assurance.

The reported data are largely misinterpreted and the conclusions distinctly misleading. In extenuation, it is only fair to add that the same misconceptions and wrong interpretations have appeared in the published results of several other costly State highway investigational projects. They are also apparent in at least one current Government investigation of magnitude if the writer is correctly informed as to procedure followed.

In view of this, Mr. McKesson's paper is timely as a basis for discussion. In the writer's remarks, the general contentions and objections will be stated; then various points appearing in the paper will be taken up separately, after which the basis for the writer's assertions will be briefly outlined in conjunction with certain curing data. At the end a suggested procedure will be given for determining how long concrete should be wet cured.

The General Contentions.—

1.—Wet concrete has been called dense concrete and reduction in weight by drying has been mistaken for lack of density.

2.—Strengths due to drying out and those due to curing have been hopelessly confused.

3.—Relative efficiencies of curing methods have been rated on short-time strength tests (mostly 14, 21, and 28-day periods), which give more of a measure of strength due to drying than that due to curing. The few 3-month tests given are likewise on specimens all the way from the saturated to the thoroughly dried out condition at test. Factors mentioned under Contentions (1) and (2) complicate both short-time and long-time curing results but exercise a much more important relative effect on the former.

4.—It has been assumed that the concrete has been sufficiently cured when test cores show a strength of 2 000 lb. per sq. in., or better. The presumption is that 2 000-lb. concrete has then been attained. The core strengths, as found in all these tests, are the strengths of dry concrete. A soaking rain reduces the strength 30 to 40%, and, further, produces a moist sub-grade that makes the loading condition far less favorable than it was with a dry sub-grade under dry concrete. This latter condition is the logical one to

use in deciding just what minimum strength should be developed before curing ceases. All exposed concrete is potentially wet concrete and no strength above that for the saturated condition can be safely assumed.

5.—Some of the curves show an unwarranted lack of agreement with the points that are supposed to define them. Other minor points will be mentioned in passing.

In Fig. 3, the smooth curves shown are supported by relatively few and very scattered groups of points. The middle curve, especially, is pulled far from its orbit by one lone satellite. It seems far-fetched to attempt to define any regular relation for such sparse and scattered plotted evidence. In offering this criticism, no attempt is made to state whether or not the curves drawn approximate the truth. As a matter of fact they do agree in general trend with the writer's findings based on much more consistent evidence. The point is that for the evidence shown, a great number of widely variant curves might be drawn, some of which could make a better claim to correctness than any of those shown. The same is true of the lower curve of Fig. 1.

The first column under "Cores" of Table 1, gives what purports to be "percentages of voids" in the cores. It is evident that these so-called voids represent approximately the volume of water that these cores would absorb. Voids and apparent specific gravities of many stones and some other materials can be found in this manner and useful constants may be obtained thereby. These materials, however, take up and lose a definite quantity of water. They can be readily dried to constant weight.

Concrete does not fall in this class of materials. Tests made at the Iowa State College,* and fully supported, in these respects, by tests by the writer, indicate that it is a rather long and tedious process to dry to constant weight. Absorption, on the other hand, is rapid, and nearly constant weight may be attained in a few hours or a day of soaking. Exposed to dry air concrete will continue to lose moisture for weeks. Placed in an oven concrete will require a long time to reach constant weight at any one temperature. Increase the temperature, and it will lose more weight. Remove it from the oven, and in even relatively dry air, it will take on appreciable weight by re-absorption of water from the air. It loses water much less rapidly after a period of moist storage than if never subjected to moist storage. At present, the writer has companion specimens in dry air (relative humidity rarely above 30% and never above 50%), some of which are taking in moisture (having been oven-dried at 70° cent. (158° Fahr.) for several weeks, but not quite to constant weight). The others have been in the air the entire period and are still losing weight.

The water in concrete is present in at least two different states. Part of it is in chemical combination and part occupies the pore space and capillary passages. The smaller passages have a strong attraction for the water and give it up reluctantly. Just when chemical separation starts the writer would not venture to say, but it is apparent that there is no abrupt

* "Effects on Concrete of Immersion in Boiling Water and Oven Drying," by W. J. Schlick, M. Am. Soc. C. E., *Bulletin 59*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

change in condition as to loss of water for temperature considerably higher than the boiling point.

All these difficulties might have passed unnoticed had the procedure been one of soaking without re-drying; and this is probably what was done. The volume of water necessary to saturate the concrete was in that case a function of the dryness of the core or other specimen at time of immersion. In either case the figure obtained has little significance and is certainly not a constant for that concrete.

Talbot and Richard* have defined the voids in concrete as the total space in a unit volume of freshly mixed concrete, not occupied by solids (cement, sand, or stone). In like manner, the density is defined as the ratio of absolute volume of solids to the total space occupied by the freshly mixed concrete. To use the term "voids" in the sense that it is used in Table 1, is not only misleading but betrays either an ignorance of accepted practice or a carelessness that is unjustified.

Concretes that were alike at mixing will continue to have equal voids and densities. One may dry to a weight of 142 lb. per cu. ft. while the other is saturated and weighs 146 lb. per cu. ft. In the former, part of the water voids have become air voids. The absolute volumes of cement, sand, and stone remain unchanged, however.

What Table 1 apparently shows is that the less dense the concrete, the stronger it is. This is absurd of course. What it does show is that a dry core is stronger per unit area but lighter per unit volume than a saturated core or cylinder of the same concrete. This is true. The conditions would have been reversed, however, were the cylinder dried and the core soaked prior to test. The one standard condition is to have both specimens saturated at test since it is difficult to attain equal degrees of dryness. In comparing cores with standard laboratory cured cylinders (wet cured, tested wet), the core should always be immersed for about one day prior to the test. Strength results are then quite comparable and weight results reasonably so.†

The actual densities of these concretes were probably 0.80 to 0.82, that is, from 20 to 18% voids, instead of the 9 or 10% given. These figures are reached by reconstructing the mixture in accordance with the incomplete data given in the report of parts of these curing tests.‡ The tests here reported doubtless used materials and proportions not greatly different, since the aim throughout seems to have been to approximate usual field conditions.

Data for each batch were as follows:§ Cement, 156 lb.; dry sand, 378 lb.; dry gravel, 522 lb.; water, 87; weight of wet concrete, 152 lb. per cu. ft.; and water-cement ratio, 0.84. Normally, the concrete was to contain 6 bags of cement per cubic yard in place.

The volume of water per cubic yard of concrete was then $0.84 \times 6 = 5.04$ cu. ft.; water voids $= \frac{5.04}{27} = 0.186$. The total voids consist of all the water

* *Bulletin 137*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

† Concrete that has once dried out will not re-absorb quite as much water as if it had been kept continuously wet (after one day or so). This is probably due to entrapped air that has replaced the evaporated water and that is not subsequently dislodged.

‡ *Bulletin* No. 15, Structural Materials Laboratory, Lewis Inst., Chicago, Ill.

§ *Loc. cit.*, p. 6.

in the mixture plus any air voids. It is seen that the water voids alone are twice the total volume of voids as given by Table 1. For a mixture as wet as this, air voids may be assumed at 0.01, which gives a total of 0.196, or a density of about 0.80.

A careful analysis made by assuming reasonable values for the specific gravities of aggregates by trial until a concrete weighing 152 lb. per cu. ft. was obtained, gives results nearly identical with the figures just given.*

If the specimens of Table 1 were originally from the same or similar concrete, their densities and, therefore, their voids (voids = 1 — density) were identical, or nearly so, as made. Evaporation or absorption does not alter the status. Voids consist of air plus water. Evaporation will lower the weight but it does not alter the voids. Water voids simply become air voids. A re-soaking returns air voids to water voids. Weights vary, densities and voids remain constant.

What in the paper may appear to be a more or less incidental detail or a slip of the pen, has been discussed at some length because it is the kind of incidental detail that leads to very serious misconceptions. Moreover, the author is by no means the only offender in this regard. There is a very general tendency to compare indiscriminately field concrete specimens of varying degrees of dryness with strengths and weights of wet tested laboratory concrete. This may be actual or by inference. Even if there are no actual laboratory specimens, the field specimens show up extra well because of dryness at test, and every one is pleased. A Government efficiency project of considerable magnitude appears to be at the present time staking its results on this same fallacy.

Dry concrete is, however, always potentially wet concrete in a pavement or other exposed construction. An hour or two of soaking rain and the extra strength is gone. Moreover, with a wet sub-grade the condition is far worse than when the sub-grade as well as the pavement is dry. Similar statements were made earlier in this discussion. They cannot be too often repeated or too strongly emphasized.

The one point of value in Table 1 is from the data in the column entitled "Strength Ratio", which gives an excellent comparison of relative strengths of similar concretes dry at test *versus* wet. Moreover, these values are exactly representative of the true difference as will be shown by the plotted test results (Figs. 7 to 13, inclusive).

The criterion for length of moist curing is given by the author as the period necessary for the concrete to develop sufficient strength to sustain traffic. This is entirely sound except that Mr. McKesson's standard of strength is that of dry concrete on a dry sub-grade. When once dried out concrete ceases to gain further strength. The curing will resume, as Mr. McKesson has stated, when the rains come. This resumed curing is, how-

* On p. 143, it is stated that the water used in these tests was about 8% of the total weight of the cement, sand and gravel. The figures used from *Bulletin No. 15*, of the Structural Materials Laboratory, Lewis Inst., would make the water $= \frac{87}{156 + 378 + 522} = 8.2\%$, which agrees closely. The concrete described at the beginning of Table 2 (p. 146), was of essentially the same proportions, having 6 bags of cement per yd.

ever, at a very much slower rate than the original.* There will be a considerable period, therefore, when the strength of the pavement is only that of a saturated concrete on a moist sub-grade. This is the critical period in the life of the pavement and this is the condition that should be the criterion for the curing period and method.

It is not here urged that the period of moist curing be unduly prolonged. Detours and curing are costly. It is not for the writer to say what is justified in a particular case. His sole contention is that correct data should be used in reaching a decision. In some cases it might well be advisable to increase the richness of the mixture in order to decrease the curing period. In other instances traffic might be turned directly on the moist earth covering or adequate sprinkling maintained under traffic. These points will be considered subsequently in detail.

The references to water necessary for given length or periods of curing (pages 142-143), indicate a somewhat peculiar point of view. There is little doubt that any workable concrete mixture contains more than enough water to satisfy all curing needs for any length of time. The only reason that watering is necessary, is the practical impossibility of conserving the mixing water. The specimen dries out. Wet-mixed concrete dries out as badly and even worse than stiff-mixed concrete. It is less dense and loses water with greater ease. There is no advantage in using extra mixing water with the idea of helping the curing process.†

The writer's remarks might by implication lead one to the fallacious expedient of using excess water in mixing for the purpose of prolonging the curing period. Advocates of wet concrete die hard and are only too ready to grab at such a fine fat straw.

Mr. McKesson is correct in his statement that the loss of water is much less, in a given time, for concrete that has had a few days of moist curing, than for a concrete exposed at once to the air.

The data of Fig. 1, lower curve, throw considerable light on different curing methods as regards their efficiency in keeping the concrete moist. The points tell little as regards relative strengths. The lower the location of a given point, the more complete the drying out. The bottom points represent specimens that are quite dry and in which curing was halted early. The right-hand points (strongest specimens) are only about half dried out. The moist earth covering, although only wet for 3, 7, or 11 days, was effective in keeping the concrete moist for a long time after sprinkling ceased. These specimens (Points 7, 11, and 13) are therefore reasonably strong because of relatively long-time curing and also strong because of partial drying. On the other hand the upper points (Points 32 and 12) have dried out least.

* For example, specimens immersed in water after 28 days in dry air had at the end of 3 months the same strength as similar specimens moist cured for 28 days only. Later tests verify these figures. For further illustration note the "In @ 25" and "In @ 61 day" curves on Fig. 7. The strength drops from the *C*-curve to the *D*-curve (about 20% of the 28-day standard strength of 25 to 30% of the *C*-curve strength) at once, due to wetting. It then gradually rises by resumed curing until about 1 month later and 5 weeks later, respectively, it has attained in the wet condition, the former dry strength. Strengths equal to the 28-day standard are reached at ages of about 65 and 130 days, respectively, *Proceedings*, Am. Concrete Inst., 1926, Fig. 13, p. 424.

† *Proceedings*, Am. Concrete Inst., 1926, Table 11, p. 408; Fig. 6, p. 415.

Potentially, these are by far the strongest specimens but the fact that they are wettest at test makes them show up less favorably. Thus, Fig. 1 shows strength comparisons between wet concrete at the top and very dry concrete at the bottom with many gradations between. Were these specimens all made comparable at test by immersion in water prior to test, the top point (No. 32) would remain fixed. All others would move up to the level of Point 32 and to the left distances more or less proportional to their present distance from the top. The final position would show Point 32 at the extreme right with the other points ranging to the left at about the same level, and in a more or less continuous alignment. There would be no abrupt bend in the curve. The 100% ordinate would then be that of Point 32. All points near the bottom would move much farther to the left than their present location since there would be present both the weakening effect of very brief curing and also a great weakening due to being wet instead of dry at test. They would move up by virtue of the added weight from soaking.

On the other hand the bottom point (No. 1) could be made the pivot by bringing all other specimens to the same approximate degree of dryness. Then the migration would be to the right and the relative arrangement identical with that just described, except for elevation. Point 32 would continue to be the right-hand point of a more or less horizontal line at the lower level.

The author mentions the possible effect of wetting on temperature. The temperature of the water applied was not greatly different from that of the concrete. After duly considering the possibility of some cooling action from evaporation, after the manner of the wet bulb thermometer or South African water bag, it is scarcely probable that temperature changes due to wetting by any method whatsoever had any measurable effect on curing within the range of temperatures that existed (about 45° to 110° Fahr. from Fig. 5). A very slight shortage of water would be far more deleterious than a change of many degrees in temperature. Moreover, curing halts when a certain degree of dryness is attained. No amount of favorable temperature will produce a further gain in strength as long as moisture is lacking. The matter of temperatures and their relation to rapidity of curing is, however, a question that needs experimental treatment. The trend is known but definite numerical relationships are undefined. The writer's comment on this phase represents only his own opinion, as opposed to the similarly unsupported statement made by the author.

Some of the author's data and deductions again illustrate the oft-recurring confusion due to the extra strength of dried out concrete compared with concrete better cured, perhaps, but wet at test. The concretes of Table 2 represents highly indeterminate conditions, especially difficult to evaluate in time tests as short as 28 days and less. All specimens were kept covered with earth for 14 days, but the watering period was varied. The earth undoubtedly retained moisture sufficient to continue curing for several days after watering ceased. Lack of drying out produced strength by added curing. Drying out reduced that strength by halting the curing,

but added that due solely to drying. Without relative weight data, it is impossible to state to what extent a given strength was due to one cause or the other. The apparent superiority over the wet-cured, wet-tested, standard laboratory specimen is, however, due in all cases to the partial drying of the field specimen. Immersion of the cores a day before the test would have shown a very different set of ratios. These would give information of value.

Fig. 3 illustrates the fact that short-time strengths are practically a straight-line function of the period of moist curing to within a few days of test. These few days represent the time required for concrete to dry out appreciably in the particular climate for that particular surface volume relation. Fig. 3 shows that this period for these tests was from 11 to 16 days.

Fig. 4 shows that the drying out was much slower in the Illinois tests cited than in the California tests, which is to be expected. The usual confusion between wet and dry concrete seems to exist.

It is interesting to note in Fig. 5 that relative humidity was at times more than 90 per cent. There is no question but that concrete rather dried out and thirsty would absorb quite a little atmospheric moisture at these higher humidities. The numerical evaluation is not at present possible and it may or may not have been appreciable in its effect on strengths.

The fact that air-cured concrete always showed a low surface hardness* is to be expected. It is low-strength concrete. Moreover, the Lewis Institute report of the tests shows the highest values of surface hardness to be for the concretes having the longest periods of moist curing. This property is less influenced by the "wet-at-test" condition, for in a very short time the surface dries out sufficiently to acquire the hardness of dry concrete.

Data from Tests by the Writer.—The writer has spoken with positiveness regarding curing and its relation to compressive strength. In this territory he is certain of his ground and the general correctness of his contentions.

Complete curing series have been run on 2 by 4-in. mortar specimens and 6 by 12-in. concrete specimens. An outline control series has been run on 6 by 12-in. mortar specimens. In addition, several partial series have been conducted on several different concretes and mortars to check and verify certain phases of the problem. Some of these results are already a matter published of record.†

A sufficient amount of evidence from a later and more extensive series that corroborate the preceding findings will be incorporated with this discussion to drive home, if possible, the fact that short-time tests give no fair measure of merit of the relative curing methods and that it is not valid to compare the strengths of concretes dry at test with those wet-tested as measures of the relative excellence of the concretes in question.

* *Bulletin No. 15,* Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., Fig. 7.

† "Curing Conditions of Mortars and Concretes," H. J. Gilkey, M. Am. Soc. C. E., *Proceedings,* Am. Concrete Inst., 1926, pp. 395–436; and "Autogenous Healing of Mortars and Concretes," H. J. Gilkey, M. Am. Soc. C. E., *Proceedings,* Am. Soc. for Testing Materials, 1926.

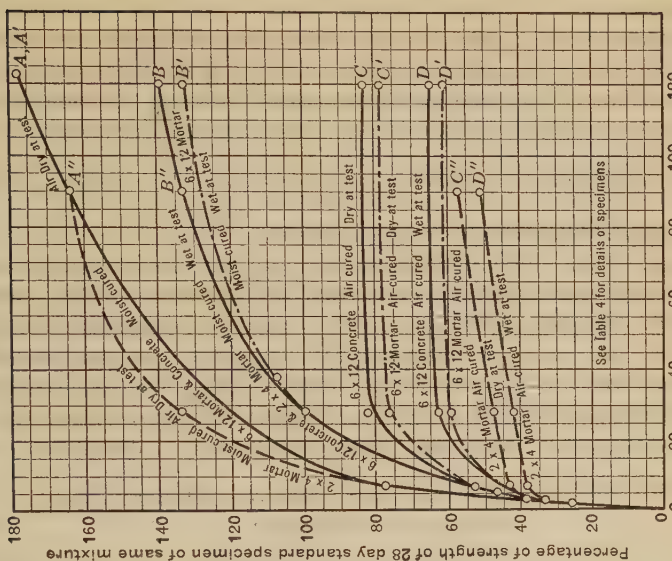

FIG. 8.—ABRIDGED UNIVERSAL CURING DIAGRAM FOR MORTARS AND CONCRETE.

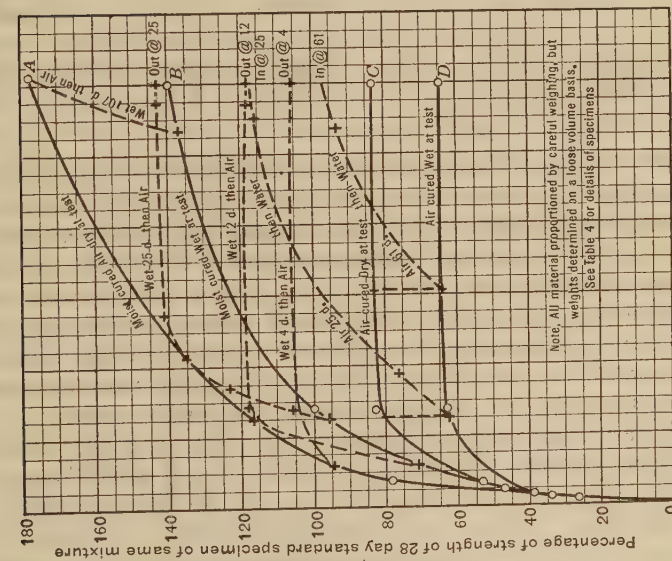
Relative Compressive Strength
Percentage of strength of 28 day standard specimen of same mixture
Age of Specimen at test in Days
A
B
C
D
Out @ 25
Out @ 12
In @ 25
Out @ 4
In @ 61
Wet 12 d. then Air
Wet 4 d. then Air
Wet 25 d. then Air
Moist cured Air at test
Moist cured Wet at test
Air-cured Dry at test
Air-cured Wet at test
Note. All material proportioned by careful weighing, but weights determined on a loose volume basis.
See table 4 for details of specimens

FIG. 7.—UNIVERSAL CURING DIAGRAM.

In Fig. 7, the *A*, *B*, *C*, and *D*-curves show a "universal" curing diagram for 1:2:3 concrete having a water-cement ratio of 0.90. The specimens were 6 by 12 in.

In Fig. 8, the *A*, *B*, *C*, and *D*-curves show the four main lines of Fig. 7. The *A'*, *B'*, *C'*, and *D'*-curves show four lines for 6 by 12-in. mortar specimens exactly like the concrete specimens except for the omission of coarse aggregate, and the *A''*, *B''*, *C*,'' and *D''*-curves show the diagram for 2 by 4-in. mortar specimens of a different mix $\left(1:2.26;\ \frac{w}{c} = 0.70\right)$. Table 4 gives comparative data for all these specimens.

The several curves in Fig. 8 show strengths all expressed as the percentage of the 28-day standard strength (wet-cured, tested wet) of that particular mixture. The strengths at all ages up to 4 months for the first two, and 3 months for the last one, are shown for (*B*), concrete, wet-cured, tested wet (standard condition); (*A*), concrete, wet-cured, but dried to approximately maximum strength and tested dry; (*C*), concrete, air-cured, tested dry; and (*D*), concrete, air-cured, tested wet (immersed in water 1 day prior to test).

TABLE 4.—TESTS FOR RELATIVE STRENGTHS DUE TO CURING. DETAILS OF SPECIMENS.

Curves.	Materials.	Size of specimen, in inches.	Proportions.			$\frac{w}{c}$.	Strength, in pounds per square inch, of standard 28-day specimen.
			Loose volume.	Weight.	Absolute volume.		
A, *B*, *C*, *D*.....	Concrete	6 by 12	1:2:3	1:2.06:2.52	1:2.48:3.18	0.90	2 950
A', *B'*, *C'*, *D'*.....	Mortar	6 by 12	1:2	1:2.06	1:2.48	0.90	3 600
A'', *B''*, *C''*, *D''*....	Mortar	2 by 4	1:2.26	1:2.5	1:2.96	0.70	4 200

The *A''*, *B*,'' *C''*, and *D''*-curves have been published.* The general trend was fully supported by various auxiliary concrete and mortar series as reported. Nevertheless, the next step in order appeared to be the construction of a similar diagram for a typical concrete mixture of 6 by 12-in. specimens, and for a longer period of time. These are the *A*, *B*, *C*, and *D*-curves shown for a 4-month period. This series will eventually cover 8 months. The jump from 2 by 4-in. mortar specimens to a 6 by 12-in. concrete series involved a change in two variables. A skeleton series of 6 by 12-in. mortar specimens for the same 8-month period was also projected, as further control.

The results are more than satisfying in the exactness with which they check the previous findings and also one another. The *B* and *B''*-curves are so perfectly in agreement that they can scarcely be distinguished. The *B'*-curve is less than 7% from the other two at greatest divergence. The *A* and *A'*-curves agree perfectly and the different shape of the *A''*-curve is exactly what should occur for the 2 by 4-in. specimens in contrast to those of 6 by 12 in. The smaller specimens dry out much faster and attain higher

* *Proceedings*, Am. Concrete Inst., 1926, Fig. 13, p. 424.

early strengths, but lower later strengths due to the earlier halting of curing action.

In the lower curves, *C* and *D* and *C'* and *D'*, the agreement is equally good for 6 by 12-in. concretes and mortars. The 2 by 4-in. specimens are more vulnerable against air-curing as they dry out sooner and a further gain in strength practically ceases. Just why the *C''* and *D''*-curves are less flat than the other similar cases is the one factor not explainable.

These several series justify the following general conclusions:

1.—Mortar and concrete specimens of similar proportions and sizes respond to curing conditions in identical manners and in essentially equal degrees.

2.—Specimens of different size respond to moist curing in exactly the same manner and extent. For other treatments, involving drying out either before or after moist storage, their relative strengths are effected by the variation in the rate of drying.

3.—Specimens of the same size subjected to environments different in drying properties would be affected in exactly the same relative manner as specimens of different sizes subjected to the same atmospheric environment.

Fig. 7 fully supports the following additional conclusions previously enunciated:*

4.—Concrete cured wet will, on removal from water, take on at a relatively rapid rate considerable additional strength, from 20 to 45% in these tests. The time to attain maximum strength is a function of the relative dryness of the atmosphere, the relation of exposed surface of specimen to its volume, etc.

5.—When concrete has reached a certain degree of dryness, further gain in strength with increased age practically ceases. The strength remains at a standstill as long as the concrete is kept dry. This means that the relative virtue of dry concrete in comparison with concrete that is continually wet diminishes with age as the wet concrete takes on added strength along Curve *B* for an indefinite period. The dry concrete is stationary; that is, the curve is horizontal, or practically so.

6.—A short-time soaking of any concrete of dry strength along either Curve *A* or Curve *C* will at once lower the strength to that of wet concrete along Curve *B* or Curve *D*. This has been duly verified by test. The phenomenon is entirely reversible. Strengths can be lowered or raised at will by soaking or drying. After each soaking there is, of course, some increase in strength due to resumed curing in the moist interval.

Figs. 9 to 13, inclusive, are derived from the universal diagrams (Figs. 7 and 8). When once the diagram is properly constructed and adequately controlled, any real or assumed curing condition may be investigated. The figures for "Apparent Relative Strength" (broken lines) represent, in all cases, the ratio of the strength as shown along the "out" curve (Fig. 7) to the corresponding ordinate of Curve *B*. This is merely an apparent strength since all strengths along Curve *B* are wet strengths. The "out"-curve strengths

* "Curing Conditions of Mortars and Concretes," H. J. Gilkey, M. Am. Soc. C. E., *Proceedings*, Am. Concrete Inst., 1926, pp. 395–436.

are all dry strengths after the first few days subsequent to removal from the water. Specimens remained somewhat moist for a few days.

The "Actual Relative Strengths" (solid lines) are the "out"-curve ordinates (Fig. 7) divided by the *A*-curve ordinates. In other words the ratio of dry strength to dry strength may be used. The same result could be obtained by immersing the "out" specimen a day before test and taking the ratio of this strength to the standard wet strength shown along the *B*-curve.

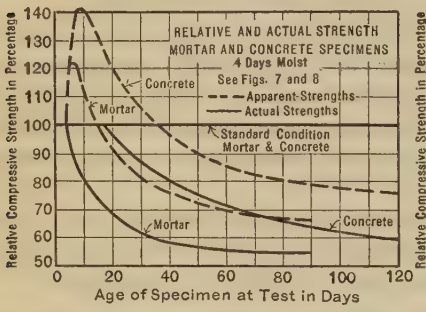

FIG. 9.

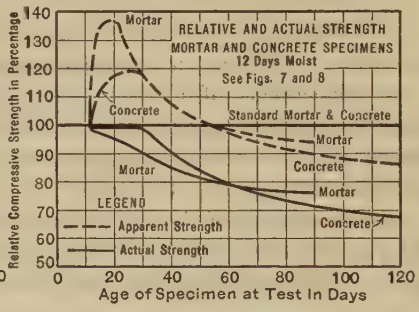

FIG. 10.

Fig. 9 shows the relative and actual strengths for 6 by 12-in. concrete and 2 by 4-in. mortar specimens removed from the water and exposed to the air after 4 days of age (3 days in water). Figs. 10 and 11 give the same information for longer periods of moist storage, namely, 12 and 25 days, respectively. All the curves give these data for both series, *A*, *B*, *C* and *D* and *A″*, *B″*, *C″* and *D″*. In other words the two similar sets of data should be an aid in evaluating the effect of varied rates of drying out.

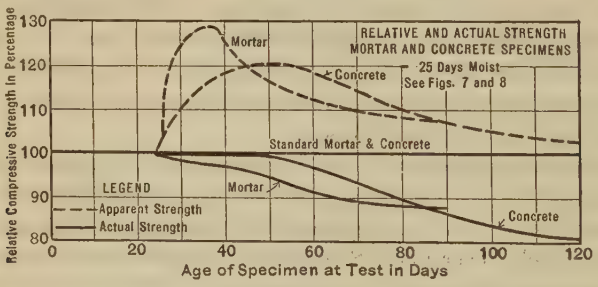

FIG. 11.

The upper part of Fig. 12 shows the apparent strength of dry concrete in terms of wet, and on the lower part the apparent strength of wet (standard-cured) concrete is shown in terms of that wet-cured but dry at test. These are all apparent strengths. Actual strength ratios are all along the 100% line. The dry concrete above, if wet, would fall to this line, whereas the wet concrete below, if dried, would rise to it.

Fig. 13 affords a check on some of the previous assertions. If the ratio of dry strengths to dry strengths is identical with that of wet strengths to wet strengths, then $C : A :: D : B$. (This, of course, refers to the respective ordi-

nates of the curves.) These two ratios show remarkable agreement for both the concrete and the mortar.

It will be noted in Fig. 9 that concrete out at 4 days gives apparent relative efficiencies as follows:

Test age (days)	7	14	21	28	60	120
Apparent relative strength (percentage of standard specimens wet at test)	141	136	116	105	87	76
Actual relative strength (percentage of standard specimens dry at test)	100	100	94	88	71	59

The actual strength ratio of 100% at 7 and 14 days simply indicates that until that time the specimen had not yet dried out sufficiently to halt curing.

Notice that the trends of both actual and apparent strengths are parallel and downward. Had longer time tests been conducted in the California investigation, the fallacy of basing conclusions on comparisons at ages of 28 days or less would have appeared in spite of the fact that strengths of wet and dry concrete were indiscriminately compared.

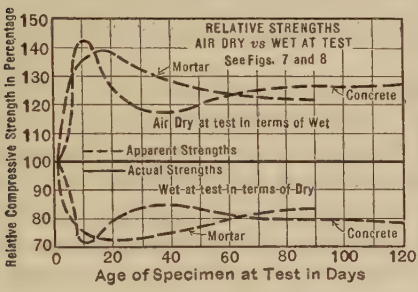

FIG. 12.

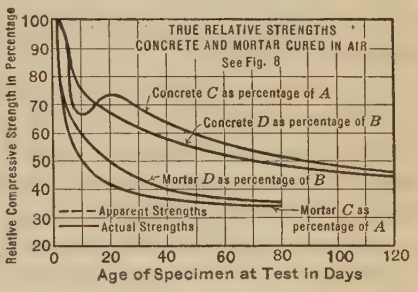
TRUE RELATIVE STRENGTHS
CONCRETE AND MORTAR CURED IN AIR
See Fig. 8
Concrete C as percentage of A
Concrete D as percentage of B
Mortar D as percentage of B
Apparent Strengths
Actual Strengths
Mortar C as percentage of A
Relative Compressive Strength in Percentage
Age of Specimen at Test in Days

FIG. 13.

It will be noted in Fig. 11 that even 25 days of moist curing shows an efficiency of only 80% that of continuous standard moist curing at the end of 4 months. It is apparently 120% efficient at 50 days. Moreover, the small mortar specimens, through early drying, because of difference in size, seem to be at a disadvantage at first. Within the time limit of these tests the concrete has in several instances already overtaken the mortar in its downward trend.

These curves will bear careful study. It is hoped that they will aid in sufficiently clarifying the subject of curing to prevent a repetition of the four major fallacies so evident in the interpretation of these tests, namely:

1.—Indiscriminate comparisons of strength of dry and wet concretes as criteria of relative curing, or any other, efficiencies.

2.—Basing conclusions as to relative efficiencies of curing methods on the results of short-time tests which fail to distinguish adequately between the curing and drying that are both present.

3.—Confusion of wetness with density.

4.—Assuming the strength of dry concrete supported on a dry sub-grade as a proper basis for deciding when curing may be safely halted.

Conclusions and Recommendations.—If the handling of Mr. McKesson's data be as seriously at fault as claimed, then it must follow that his conclusions are probably of little value. What then shall be substituted for the three simple recommendations (page 150), that appear to be such a happy and usable ending?

First, it must be recognized that the most severe test in the life of the pavement will not necessarily come with the first turning on of the traffic. Nor is there much question relative to an old pavement.

When traffic is turned on, the concrete may be quite dried out and the sub-base firm. On the other hand old concrete will have passed through several alternating periods of dryness and wetness. Curing halts during dry periods but resumes at a slow but appreciable rate during wet periods. Thus, any incomplete early curing is remedied in the course of time and the concrete gets better and better with age. Even a little moisture retained in the sub-grade is a great aid to the concrete in so far as curing is concerned. The real test comes when the concrete gets its first rewetting on a softened sub-grade. The resumed curing is at altogether too slow a rate to help materially in this particular emergency.

First, then, a decision must be made as to what strength will be needed to tide the pavement over this period. That decided, this strength should be expressed as a percentage of the 28-day standard strength of the concrete being used (this means the wet-cured, wet-tested, 28-day strength). Then reference to Curve *B* (Fig. 7 or Fig. 8) will indicate what period of wet curing is necessary to develop about that percentage of the 28-day standard strength.

During reasonably dry weather, traffic may be turned on before curing is complete. The sub-base is firm even if the concrete be kept moist for curing. The curing should be kept up, simultaneously with use, however, in preparation for that more critical period when both pavement and sub-grade are wet. For example, if 2 500-lb. concrete is being used and it is decided that it should have a strength of 2 000 lb. before being placed in service, it is found that about 80% of the 28-day standard strength will be developed in 16 days of moist curing.

Along Curve *A*, which shows both the hardening and curing, the 80% strength is attained when the concrete is about 8 days old. The concrete, if still being cured, is wet, but the sub-grade, in good weather, should be firm and the worst condition is not present. Therefore, traffic may safely be turned on somewhere between 8 and 16 days. However, in preparation for the evil day to come, when the concrete will be wet, and the sub-grade soft, precaution must be taken to insure the equivalent of a full 16-day moist curing period.

This does not necessarily mean that the pavement must be kept ponded or covered for the full 16 days. In a truly dry climate this should doubtless

be approximated. In climates less dry there might be sufficient residual moisture from 10, or even 7, days of actual soaking to enable curing to proceed without serious interruption through the full 16-day period. If the sub-grade be even reasonably moist, quite a little reliance can be placed on the moisture that it will furnish through capillarity. During dry months when the foundation is firm, traffic can be turned directly on the moist earth covering and the pavement can be used while it is also being strengthened against the future test. If a moist earth covering is not used, added strength can be attained by adequate sprinkling while in service. In a windy or dry climate too much reliance, however, should not be placed on the sprinkling, as much of the water is lost in a short time by evaporation. In many cases it would undoubtedly be excellent economy to use a richer mixture in order to reduce "out-of-service", detour, and curing costs.

There are two or three points in which the element of judgment must enter to an objectionable degree:

(*a*) Just what strength will be sufficient to tide over the emergency period is a problem involving the nature of the sub-grade and the still unsolved problem of how a yielding sub-grade supports a rigid pavement under more or less concentrated loading. That problem is no more an inherent part of the situation than it is to the whole matter of suitable design stresses, pavement loading, etc. This is still a question for the investigator and expert analyst.

(*b*) How long curing will continue after direct application of water has ceased, is a real problem for the field man. By careful and intelligent observation he should be able to judge this quite satisfactorily. A moist earth covering left on is one of the most effective methods for retention of moisture.

(*c*) Curve *B* may not be indicative of the time rate of gain in strength for all concretes, but it is typical and doubtless closely approximates most cases of standard curing. A better curve of course could be obtained from a few specimens of the concrete in question.

Barring poor construction, including the placing of concrete pavements on loose fill, insufficient allowance for expansion and contraction, etc., it is probable that initial or incipient failure usually dates to the critical period when relatively young saturated concrete is being called on to support loads on a soft sub-base.

Late fall pavement, with retarded winter curing because of low temperature, is very likely to suffer serious damage during the spring thaws. This case is not of interest in California, however.

Acknowledgment.—The writer is indebted to William H. Thoman, Instructor at the University of Colorado, for help in preparing the data.

C. L. McKesson,* Assoc. M. Am. Soc. C. E. (by letter).—The discussion by Professor Gilkey shows how readily different investigators may find apparent support for divergent theories in data of the same and similar tests. He criticizes at length what he designates as the misuse of the term "voids" in the paper. This term was used in Table 1 which was a part of a description

* Materials and Research Engr., California Highway Comm., Sacramento, Calif.

of a series of tests made to determine the relative efficiency of various methods of tamping and compacting concrete and had no connection with the design or curing of concrete.

Professor Gilkey asserts that the use of the term betrays "either an ignorance of accepted practice or a carelessness that is unjustified." It was neither. The writer is familiar with the published reports of splendid work done by Talbot and his associates and of the special definition which they give to the term "voids" in connection with their design formula. The term was used in its most commonly accepted sense; it could not easily have been misunderstood and the writer believes that such use was quite proper. That the thought intended to be conveyed was very clearly expressed is evident from the fact that Professor Gilkey correctly understood it as "approximately the volume of water which these cores would absorb." The restricted significance of the term as sometimes used in the design of concrete would have rendered it meaningless in connection with the tamper tests described. Inasmuch as the term did not relate to the consideration of concrete curing to which his discussion is otherwise entirely devoted further reference to its use or "misuse" appears purposeless at this time.

He believes that concrete curing tests conducted by the California Highway Commission and the co-operative tests of the California Highway Commission and the Lewis Institute are examples of careful and costly work largely neutralized because of misinterpretation and incomplete analyses. He further states that the same misconceptions have appeared in the reports of several other costly State highway and Governmental investigations, and asserts that he "feels competent to speak with assurance". His comments are, therefore, in the form of statements of fact rather than his expressions of opinion.

The problem of proper curing of concrete is one of considerable importance and Professor Gilkey's viewpoint, if correct, would tend to overthrow the data and conclusions already compiled from a number of studies conducted, as he states, at the expense of the public. It seems desirable, therefore, to analyze his statements with considerable care.

Concrete curing studies have been conducted by several States. Outstanding among these is an elaborate study made by the Illinois Highway Commission and reported by Clemmer and Burggraf.* Another carefully planned research is now being conducted by the U. S. Bureau of Public Roads. Preliminary results of the latter study were reported by Mr. J. T. Pauls† at the December, 1926, meeting of the National Research Board. It is significant that results obtained in these investigations appear to justify the conclusions based on the California studies, as presented by the writer. In fact, the report by Mr. Pauls includes curves which could be fully supported by the California test data.

With reference to the curing tests Professor Gilkey asserts that "wet concrete was called dense concrete and a reduction in weight by drying was mistaken for lack of density." The writer knows of no such statement. The

* *Proceedings*, Am. Soc. for Testing Materials, Vol. 3, 1923, p. 2.

† *Public Roads*, December, 1926, p. 202.

only time "voids" or "density" were referred to directly or indirectly was in connection with Table 1 which had to do solely with tamper tests. Professor Gilkey states that strengths "due to drying out and those due to curing are hopelessly confused." The writer believes this statement to be entirely incorrect; on the contrary, the paper was most explicit on these points.

In the report of the 1924 series* the loss of water in every specimen from the time of fabrication to the time of test was determined and carefully noted. In 28 and 90-day tests the only specimens that were tested wet were the beams, which had been kept wet for 26 and 88 days. It was pointed out that these beams had an increase in moisture content of 5.7% and 10.4%, respectively (expressed in terms of original mixing water); and that, as a result of free moisture present at the time of test, they gave lower strengths than the beams that had been kept wet only 7 and 14 days and then had been allowed to dry for the remainder of the 90-day period. The report explicitly stated that concrete tested in a damp condition "is weaker than similar concrete tested dry."

The writer further emphasized the statement that wet specimens have less strength than the same specimens would have if dried by referring to the studies by Professor Hatt,† which show reductions in strength due to the presence of free moisture in concrete at the time of test.

Effect of Moisture on Strength of Concrete.—In order that there may be no confusion in the mind of any one regarding the effect of various curing conditions on moisture content of concrete a detailed discussion of this phase of the studies seems desirable.

Fig. 1 shows moisture changes in beams during the first 90 days after moulding. Moisture changes at ages up to and including 90 days, have also been published.‡ Moisture content is expressed as a percentage of the original mixing water. It was determined by weighing the mixing water and all dry ingredients, by weighing the specimens when removed from forms the morning after moulding, and by weighing again prior to testing. The specimens were 7 by 10 by 38-in. beams. They were placed on asphaltic building paper to prevent moisture escaping to the sub-grade, and the paper was brought up around the sides of the specimen. The upper surfaces of the beams were subjected to various curing treatments, including watering for 3, 7, 14, 28, and 90 days. Table 5 shows the moisture changes expressed in terms of mixing water and in terms of total weight of specimen, as well as flexural strengths.

Table 5 is compiled from data given in the reports previously published, and it is believed that a careful study of it will prove worth while. Several features seem to be outstanding, the most important being the great difference in moisture losses between specimens exposed in the air and those watered even such a brief time as 3 days. The air-cured specimens lost 15.2% of their moisture content in the first 3 days and continued losing moisture until the total reached 32.7% in 90 days. Three days of watering apparently reduced this 90-day loss by more than three-quarters. Specimens watered 7 and 14

* "Studies of Curing Concrete in a Semi-Arid Climate," by H. F. Gonnerman and C. L. McKesson, Assoc. M. Am. Soc. C. E., *Bulletin 15*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

† "The Effect of Moisture on Concrete," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 270.

‡ "Concrete Curing Tests," Table 4, *Bulletin 15*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

days showed no difference in water content in 90-day tests. The writer believes that this apparent tendency for water to unite with cement, probably in some loose chemical combination, during the first days of curing is responsible for the very satisfactory showing made by short periods of watering in these (1924) curing tests and also in the 1925 curing tests.

TABLE 5.—Moisture Content Changes in Concrete and Modulus of Rupture. Various Ages and Conditions of Curing.

Age at test, in days.	Test.	Air. No cover. No water.	2-Inch Earth Cover Wet as Indicated, Then Removed. Remainder of Time in Air, No Cover.				
			Wet, 3 days.	Wet, 7 days.	Wet, 14 days.	Wet, 26 days.	Wet, 88 days.
3	Change in moisture, percentage of mixing water.	−15.2	+3.3	.....	.....	.....	.....
	Change in moisture content, percentage of weight of specimen.....	− 1.1	+0.3	.....	.....	.....	.....
	Modulus of rupture.....	295	340	.....	.....	.....	.....
7	Change in moisture, percentage of mixing water.	−18.0	+1.6	+6.6	.....	.....	.....
	Change in moisture content, percentage of weight of specimen.....	− 1.4	+0.1	+0.5	.....	.....	.....
	Modulus of rupture.....	345	435	445	.....	.....	.....
14	Change in moisture, percentage of mixing water.	−22.8	−4.4	−3.3	+5.5	.....	.....
	Change in moisture content, percentage of weight of specimen.....	− 1.7	−0.3	−0.3	+0.4	.....	.....
	Modulus of rupture.....	365	490	480	460	.....	.....
28	Change in moisture, percentage of mixing water.	−27.2	−8.7	−4.9	0	+5.7	.....
	Change in moisture content, percentage of weight of specimen.....	− 2.0	−0.7	−0.4	0	+0.4	.....
	Modulus of rupture.....	385	505	530	540	510	.....
90	Change in moisture, percentage of mixing water.	−32.7	−7.1	−5.5	−5.5	.....	+10.4
	Change in moisture content, percentage of weight of specimen.....	− 2.5	−0.5	−0.4	−0.4	.....	+ 0.8
	Modulus of rupture.....	435	565	605	595	.....	485

The specimens wet 28 days and tested at 30 days and those wet 88 days and tested at 90 days contained an excessive quantity of free moisture and, as previously pointed out, their strength was probably reduced thereby. Professor Hatt, in the paper previously referred to, shows that saturation of specimens reduces their flexural strength 20 per cent. Saturation in the data presented by Professor Hatt consisted of the absorption of about 2% of water. In the data presented in Table 5 there is approximately 1% difference in the moisture content of specimens cured by various amounts of watering. Assuming the strength of the saturated 88-day specimens to be only 80% of that when dry, their theoretical moduli of rupture would be 606 lb., practically the same as for specimens watered 7 and 14 days. The error in this reasoning, if any, is in favor of the saturated concrete because it is probably unfair to refer to 7 and 14-day watered specimens as dry in view of the fact that they contain 2.0% more water than the air-dried specimens.

TABLE 6.—California Highway Commission Curing Tests. Compressive Strengths, 6 by 12-Inch Moulded Cylinders and 4½ by 7-Inch Drilled Cores (1925 Series).

(Concrete mix, 1 : 1.9 : 3.8, using 6 sacks of cement per cubic yard.)

Section No.	Watering period, in days.	*6 by 12-in. cylinder, at 28 days.	Age of Test.									
			14 Days.		21 Days.		28 Days.		90 Days.		1 Year.	
			† Cores, in pounds per square inch.	‡Strength ratio.	† Cores, in pounds per square inch.	‡Strength ratio.	† Cores, in pounds per square inch.	‡Strength ratio.	† Cores, in pounds per square inch.	‡Strength ratio.	† Cores, in pounds per square inch.	‡Strength ratio.
1	14	3 296	3 285	100	3 873	118	4 229	128	4 301	132	4 558	138
2	12	3 105	3 468	112	4 103	132	4 414	142	4 895	157	4 720	152
3	10	2 930	3 123	107	3 694	126	4 225	144	4 475	153	4 827	164
4	8	2 968	3 366	113	3 483	117	3 877	130	4 264	144	5 329	179
5	7	3 118	3 073	99	3 596	115	4 101	132	4 924	158	4 940	158
6	3	3 356	3 948	118	4 323	129	4 068	121	4 659	139	5 110	152
7	0	3 378	3 329	99	3 436	102	3 814	113	3 465	103	3 673	109
Average	....	3 164	3 370	106	3 787	119	4 104	130	4 426	141	4 737	150

* 6 by 12-in. cylinders made in accordance with American Society for Testing Materials standards, and cured in water for 28 days. Each strength an average of ten specimens except Section 7, in which only four specimens were made.

† Cores drilled with Calyx shot drill. Cores tested were free from visible defects. Each strength is an average of 5 cores.

‡ Strength ratios are ratios of the core strengths to 28-day strengths of the cylinders from the same sections.

Additional Test Data.—Professor Gilkey states:

"Had longer time tests been conducted in the California investigation, the fallacy of basing conclusions on comparisons at ages of 28 days or less would have appeared in spite of the fact that strengths of wet and dry concrete were indiscriminately compared."

Fortunately, the tests were continued; further, the results at 90 days and 1 year fully justify the original deductions. These tests are a continuation of the 1925 Sacramento curing tests fully described in connection with Table 2.

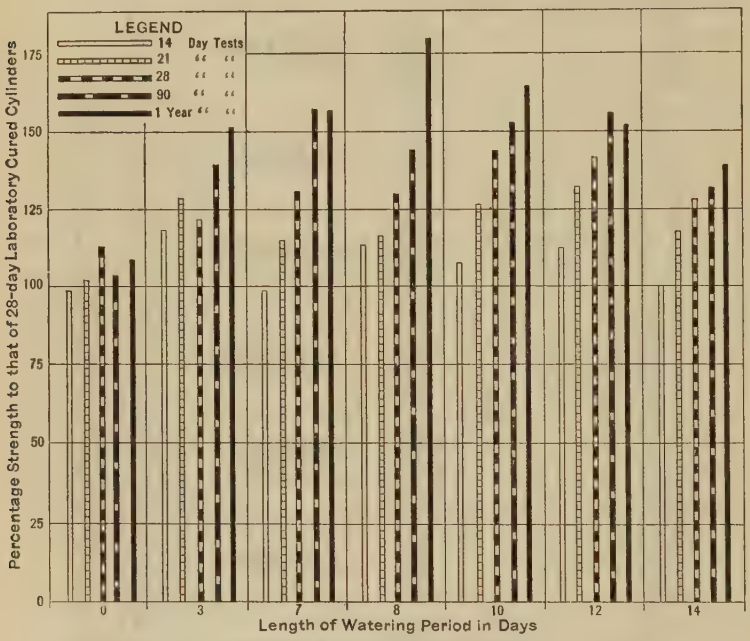

FIG. 14.—CALIFORNIA HIGHWAY COMMISSION, FIELD TESTS, CURING CONCRETE PAVEMENT, 1925 SERIES.

All sections of the pavement had the earth cover removed a t14 days. The 90-day tests consisted of cores drilled from the pavement in August, 1925, after the pavement had been exposed 76 days in the sunshine. The daily temperature in the sunshine probably averaged well above 100° Fahr., and there was no rainfall. It is, perhaps, safe to assume that all the specimens were air-dried in so far as they would thus yield moisture. The 1-year specimens were drilled in May and June, 1926, several weeks after the close of a rainy season, which showed a total of about 17 in. of rainfall.

Table 6 gives the results of the 90-day and 1-year tests as well as of those made at shorter periods.

The results of these tests are also shown in Fig. 14. The writer feels that the best discussion of these new test data is to point out that they apparently fully verify the original conclusions as follows.

Conclusions.—

1.—That seven days of wet earth curing (or watering) is ample to provide moisture for hydration through a curing period of 28 or 90 days.

2.—That this watering period might be reduced to 3 days where concrete is of high-class quality and where a substantial saving can thus be effected.

3.—That during summer weather concrete pavement may be opened to traffic in 14 days. (During cold weather the hardening of concrete is retarded and the time of opening should depend on the actual conditions in each case).

All the sections of pavement involved in the 1925 tests have now (March, 1927) been in use under heavy traffic for 20 months and none of them shows evidence of weakness as a result of under-curing. The section that received no watering has a softer surface as was demonstrated during the summer of 1926 by a ball indentation test; but abrasive losses have not yet been noticeable. All these sections were cured under extremely unfavorable conditions of high temperatures, low humidity, and lack of rainfall.

The writer will not attempt to explain why 7 and 8-day watering shows consistently higher strengths than 14-day watering, but again suggests that higher early temperatures may have played a part in producing this condition. Professor Gilkey doubts this, but the writer would point out that dry-cured specimens in the 1924 tests* maintained a temperature averaging about 18° Fahr. higher than specimens which were kept covered with wet earth. This early higher temperature of the specimens allowed to dry on the surface after 7 or 8 days may have an appreciable influence on the strength of the concrete.

Professor Gilkey in support of his contentions cites results of a very extensive laboratory series of curing tests which he has made, using 2 by 4-in. and 6 by 12-in. specimens. Conditions in such laboratory tests and under the field conditions described in the paper are very different and the writer's experience does not indicate that the results can be expected to be comparable. The pavement is placed on a dampened sub-grade. A portion of the mixing water doubtless passes into the sub-grade when the pavement is laid, to be returned later by capillarity when the pavement becomes dryer than the sub-grade. A protective layer of earth over the pavement aids in preventing early evaporation of moisture. Apparently, it is generally agreed that the first few days are the most important in the curing period.

The writer does not believe there are any conflicts between the test data produced by Professor Gilkey and those presented in the paper and in this discussion, that cannot be reconciled by differences in conditions under which the tests were made. Inasmuch as the California tests were practically duplicated with consistent results during two successive summer seasons and under extremely unfavorable climatic conditions the writer does not feel called on to attempt an analysis of Professor Gilkey's tests with the idea of attempting to point out further the reasons for apparent differences which he believes are fully reconcilable.

* *Bulletin 15*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

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Paper No. 1625

DISTRIBUTION OF REINFORCING STEEL IN CONCRETE BEAMS AND SLABS*

By Boyd S. Myers,† M. Am. Soc. C. E.

With Discussion by Messrs. Edward Godfrey, H. C. Sandbeck, James B. Newman, E. F. Rockwood, C. A. Willson, F. Theodore Mavis, Jacob Feld, Charles S. Whitney, David A. Molitor, L. J. Larson, Anton Brandtzaeg, John J. Gould, and Boyd S. Myers.

Synopsis

The object of this paper is to determine the correct amount of reinforcing steel required in the top of concrete beams and slabs at the supports for varying degrees of continuity. The discussion is based on the established fact that the area of steel required in a simply supported concrete beam is determined by the equation, $M = \frac{W\,l^2}{8}$, from which, by assuming various degrees of continuity, the area of steel in the top, over the supports, is found. The area of steel in the bottom at mid-span plus the average area in the top, over the supports, is shown to be equal to the area required in the bottom of a simply supported span of the same length carrying the same load.

Many city building codes and other recommendations require the total area of steel in concrete beams to be calculated from the equation, the moment, $M = \frac{W\,l^2}{6}$. The writer contends that this moment should be $M = \frac{W\,l^2}{8}$, giving an area of steel that will adequately and satisfactorily care for all the bending stresses.

* Published in August, 1926, *Proceedings*.

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General Principles

Although reinforced concrete structures have been used extensively for the past twenty-five years, and the general theory of their design has been quite fully developed, yet engineering literature, including the publications of the Society, give meager discussions of the proper distribution of reinforcing steel in concrete beams and slabs. Believing that this subject is as important as the correct area of steel to be used, the writer presents this paper in the hope that the discussion will lead to the establishment of the correct constants to use in the design of monolithic concrete structures.

The following general conditions should be kept in mind:

1.—Concrete beams and slabs are mechanical compositions of sand, cement, rock, and steel, and, consequently, are non-homogeneous.

2.—They usually have T-sections at the center of the span, and rectangular sections at the supports, and, therefore, have varying moments of inertia.

3.—Therefore, they cannot readily be analyzed by the three-moment theorem or similar theory.

4.—In monolithic concrete structures it is permissible to consider slabs and beams as fixed at their supports. An interior beam framing into a column at each end, in line with similar beams in the adjacent spans, unquestionably is fixed at its supports. A single-span beam framing into concrete columns should be considered as partly fixed at the supports, as will be shown later.

Likewise, a beam framing into girders instead of columns with beams of equal or greater size in the adjacent spans should be considered as fixed at the supports. If the beams in the adjacent spans are smaller, however, the interior beam should be considered as partly fixed, as will be developed later. It will be assumed that concrete beams in monolithic structures are fixed or partly fixed at their supports and the bending moments produced at the supports, based on certain reasonable assumptions, will be determined.

Simply Supported Beams

Let $B\,B$ (Fig. 1) represent a simply supported beam of span, l, carrying a uniform load of W lb. per lin. ft. Evidently, the equal reactions at the supports, B and B, are $\frac{W\,l}{2}$. It is also evident that the maximum bending moment occurs at the center line of the span and is equal to:

$$M_c = \left(\frac{W\,l}{2} \times \frac{l}{2}\right) - \frac{W\left(\frac{l}{2}\right)^2}{2} = \frac{W\,l^2}{8}$$

The variation of the bending moment may be shown graphically by a parabola, $B\,C\,B$ (Fig. 2), drawn through the points of support, B. The vertical ordinates from the base line, $B\,D\,B$, to the parabola, $B\,C\,B$, shows the intensity of the bending moment at any point along the span length of the beam, $B\,B$.

Beams Fixed at Both Supports

Assume that instead of being simply supported, this beam, *B B* (Fig. 2), is fixed at the supports. Evidently, there will be a bending moment at the support inducing a zone of tension in the top of the beam and a zone of compression in the bottom. Then, as there is tension in the bottom at the center

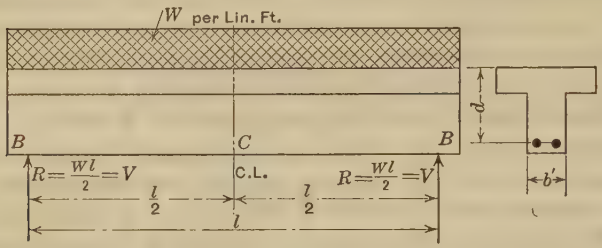

Fig. 1.

line of the span, and compression in the bottom at the support, there must be a point of zero stress somewhere between the center of the beam and the support. This point near each support is generally known as the point of contraflexure. The part of the beam between the points of contraflexure may be considered as a simple span, and the part from the point of contraflexure to the support, as a cantilever beam.

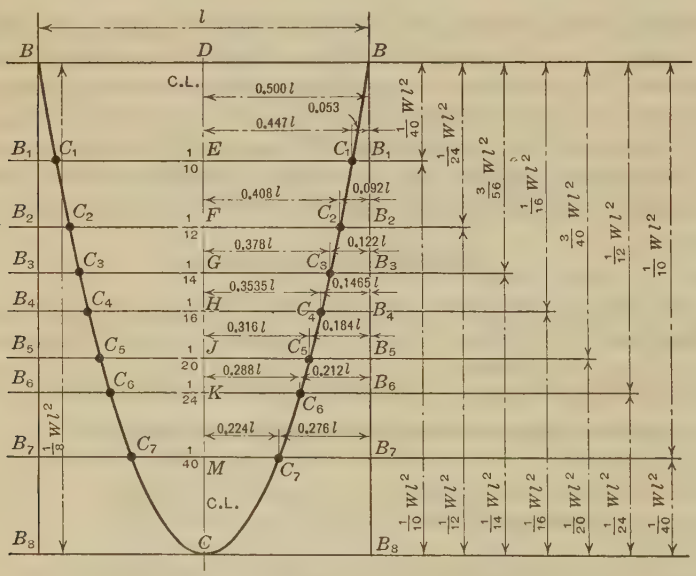

Fig. 2.

Assume that the bending moment at the center of this beam, *B B*, is $\frac{W l^2}{10}$, locate the points of contraflexure and determine the bending moments

which are produced at the supports. Referring to Fig. 2, draw the base line, $B_1\ E\ B_1$, in such a position that $EC : DC = 8 : 10$, or $EC = 0.8\ DC$. Then, the parabola, $C_1\ C\ C_1$, represents the bending moment between the points of contraflexure. Considering the base line, $B\ B$, as unity, the distance between the points of contraflexure may be determined as follows:

$$(C_1\ C_1)^2 : (B\ B)^2 :: 8 : 10$$

$$C_1\ C_1 = \sqrt{0.8\ (B\ B)^2}$$

but,

$$B\ B = l = \text{unity}$$

therefore,

$$C_1\ C_1 = \sqrt{0.8} \times l = 0.894\ l$$

Then, the distance, $E\ C_1$, from the center line to the point of contraflexure is $0.447l$, and the distance, $B_1\ C_1$, from the point of contraflexure to the support is $0.053l$. Now, considering $C_1\ C_1$ as a simple span carrying the load, W, per linear foot, the reaction at C_1 is $0.447Wl$; then the moment produced on the cantilever beam, $C_1\ B_1$, is,

$$(0.447\ W\ l \times 0.053\ l) + \frac{W\ (0.053\ l)^2}{2} = 0.0236\ W\ l^2 + 0.0014\ W\ l^2$$

$$= 0.025\ W\ l^2 = \frac{1}{40}\ W\ l^2$$

It will be noted that the sum of the moment at the center of the span and the average of the moments over the supports is equal to the moment at the center of a simple span of the same length, that is,

$$\frac{1}{10}\ W\ l^2 + \frac{1}{40}\ W\ l^2 = \frac{5}{40}\ W\ l^2 = \frac{1}{8}\ W\ l^2$$

Also, that the moment at the support is one-fourth the moment at the center of the span.

In like manner, assume that the bending moment at the center of the beam, $B\ B$, is $\frac{1}{12}\ W\ l^2$, represented by $F\ C$ (Fig. 2). Then,

$$F\ C = \frac{8}{12}\ D\ C = 0.667\ D\ C$$

and, considering the base line, BB, as unity, the distance, $C_2\ C_2$, between the points of contraflexure is found to be:

$$\sqrt{0.667} \times l = 0.816\ l$$

Considering $C_2\ C_2$ as a simple span carrying the load, W, per foot of span, with a reaction at $C_2 = 0.408Wl$, and $B_2\ C_2$ as a cantilever, the moment at B_2 will be,

$$(0.408\ W\ l \times 0.092\ l) + \frac{W\ (0.092\ l)^2}{2} = 0.0375\ W\ l^2 + 0.0042\ W\ l^2$$

$$= 0.0417\ W\ l^2 = \frac{1}{24}\ W\ l^2$$

Notice, again, that the sum of the moment at the center of the span and the average of the moments over the supports is equal to the moment at the center of a simple span of the same length, that is,

$$\frac{1}{12} W l^2 + \frac{1}{24} W l^2 = \frac{3}{24} W l^2 = \frac{1}{8} W l^2$$

Assuming further values for the moment at the center, successively, as $\frac{1}{14}$, $\frac{1}{16}$, $\frac{1}{20}$, $\frac{1}{24}$, $\frac{1}{40} W l^2$ and, finally, as 0, the dimensions of simple beams and cantilevers in each case will be as shown in Fig. 2. In all cases the sum of the center moment and average moment at the supports is equivalent to the moment for a simple beam of the same span.

Since the bending moments at the supports and at the center of the span determine the area of steel required at those points, the foregoing discussion shows that the designer has a wide latitude in the distribution of the steel in the interior spans of a monolithic concrete structure. He can place in the bottom of the beams all the steel required by the equation, $M = \frac{1}{8} W l^2$, and thus take care of the positive bending stresses, but, on account of the monolithic construction, unsightly cracks would develop over the supports and near the columns and these might be serious enough to affect the strength of the structure. Again, he can place all this steel in the top over the supports and detail the beams as cantilevers to develop the resulting tension, and he would have a theoretically satisfactory design. Or, he can place any part of the steel required by this equation in the bottom of the beam at the center of the span, and the remainder in the top of the beam at the supports, detailing the beam accordingly, and the design will care for all the bending stresses adequately and satisfactorily.

Considerations Affecting Distribution

The distribution of the steel in beams and slabs should be governed by the following considerations:

A.—The area of steel in the top of a beam framing into a column should be a minimum consistent with good designing, so that the bending moment produced in the columns either by unbalanced loads, by unequal spans, or by a beam framing into one face of the column only, may be a minimum. The columns perform the primary function of supporting the floor loads and should not be burdened with unnecessary bending which would necessitate a material increase in their size and cost.

B.—In practically all cases, the beam has a T-section at the center of the span, and a rectangular section at the support. Therefore, at the support, the concrete can develop the full stress in only approximately one-half as much steel as at the center of the span. If the area of steel in the top at the support is equal to or greater than the area placed in the bottom at the center, it is necessary to increase the depth of the beam considerably

at the supports, in order to develop full stress in the top steel. This materially increases the cost of building forms and partitions to conform to the irregular and sloping soffits of the beams.

C.—Beams framing into girders from opposite sides and having approximately equal spans should have over the supports a minimum area of steel consistent with good designing. In this case, each beam is fixed by its companion, and by the torsional resistance of the girders. Evidently, the greater the area of top steel, the greater will be the torsional stresses produced by unbalanced loads, or by unequal spans.

D.—After a careful study of the foregoing theoretical and practical considerations, it should be evident that in only one type of construction is it logical and practical to place the same area of steel in the top over the support as in the bottom at the center of the span. This type consists of slabs of approximately equal spans forming parts of the slab-and-beam type of construction. The slab has the same depth at the support as at the center of the span, and, consequently, will develop the same area of steel at both points. The interior slab spans then should be designed for $M = \frac{1}{16} W l^2$. If however, the area of steel over the supports is made three-fourths of the area placed in the bottom at the center of the span, the interior spans should be designed for $M = \frac{1}{14} W l^2$. Similarly, for one-half as much in the top as in the bottom, the interior slab spans should be designed for $M = \frac{1}{12} W l^2$.

Unless a beam has a rectangular section throughout its entire length it is impractical and illogical to place the same area of steel in the top over the supports as is placed in the bottom at the center of the span. This condition is of such rare occurrence that it need not be considered. It is uneconomical to place more steel in the top over the supports than the rectangular beam section will develop; it is costly and undesirable either to extend the straight bars in the bottom of beams into the adjacent spans doubly reinforcing the beams at the supports or to build heavy concrete haunches at the supports, to develop the top steel.

The bending produced in the columns by a large area in the top of the beams at the supports is a vital factor in the design. The wall columns with beams framing into them at right angles to the wall line would be subjected to a serious bending stress. This bending stress is taken by a section of the column extending approximately a half story both below and above the beam, provided the column bars extend about 40 diameters below and above the rough floor for bond. The interior columns would experience bending from either unbalanced loads or unequal spans. The wall columns should not be subjected to a bending greater than that produced by placing an area of steel in the top at the support of about one-fourth that in the bottom at mid-span. For ordinary loads and for spans up to about 25 ft., the bending in the columns would not be serious and can be easily cared for in the design based on the foregoing assumptions.

Proper Moment Coefficients

A careful study of these theoretical and practical considerations points to the following conclusions:

1.—Single-span beams framing into brick walls should be considered as simply or freely supported, and should be designed for a moment, $M = \frac{1}{8} W l^2$, at mid-span.

2.—Single-span beams framing into concrete columns should be considered partly fixed and should be designed for moments, $M = \frac{1}{10} W l^2$, at the center, and $M = \frac{1}{40} W l^2$, at the supports.

3.—End-span beams framing into concrete columns should have one-fourth as much steel in the top at the end support and one-half as much at the interior support as in the bottom of the beam at mid-span. The principle previously developed that the total area of steel in any beam is equivalent to that required by the equation, $M = \frac{1}{8} W l^2$, applies to this case. Assuming that $M = \frac{1}{11} W l^2$ at mid-span, $M = \frac{1}{22} W l^2$ at the interior support, and $\frac{1}{44} W l^2$ at the exterior support, so that:

$$\frac{1}{11} W l^2 + \frac{\left(\frac{1}{22} W l^2 + \frac{1}{44} W l^2\right)}{2} = \frac{1}{8} W l^2$$

4.—End-span beams framing into brick walls or into concrete lintel beams should be considered as freely or simply supported at the wall end. These beams should be designed for $M = \frac{1}{10} W l^2$ at mid-span, and for $M = \frac{1}{20} W l^2$ at the interior support, whence, as before,

$$\frac{1}{10} W l^2 + \frac{\frac{1}{20} W l^2}{2} = \frac{1}{8} W l^2$$

5.—Interior-span beams should have an area of steel in the top at the supports equal to one-half that in the bottom of the beam at mid-span. These beams should be designed for $M = \frac{1}{12} W l^2$ at mid-span and $M = \frac{1}{24} W l^2$ at the supports.

6.—End-span slabs should have the same area of steel over the interior support as in the bottom at mid-span. It is common practice to bend up alternate slab bars, so the area in the top at the exterior support would be one-half the area of the steel in the bottom at mid-span. The end-span slabs should be designed for $M = \frac{1}{14} W l^2$ at mid-span, $M = \frac{1}{14} W l^2$ at the interior

support, and $M = \frac{1}{28} W l^2$ at the exterior support. Then, as recommended:

$$\frac{\frac{1}{14} W l^2 + \frac{\frac{1}{14} W l^2 + \frac{1}{28} W l^2}{2}}{2} = \frac{1}{8} W l^2$$

If the end span is built at least 8 in. into a solid brick wall or if it is built monolithic with a fair-sized lintel beam, the exterior support will develop top steel equivalent to $M = \frac{1}{28} W l^2$, unless the slab span is unusually large. If the exterior bearing is considered a simple support, the end-slab span should be designed for $M = \frac{1}{12} W l^2$ at mid-span and $M = \frac{1}{12} W l^2$ at the interior support. Again,

$$\frac{\frac{1}{12} W l^2 + \frac{\frac{1}{12} W l^2}{2}}{2} = \frac{1}{8} W l^2$$

This last case departs from a strictly monolithic construction, and a careful analysis will show that the moment constants given will be deficient by about 3 per cent. This will not affect the design materially, but, if accurate results are desired, the end-slab span with a simple exterior support should be designed for $M = \frac{3}{35} W l^2$ at mid-span and $M = \frac{3}{35} W l^2$ at the interior support.

Beams Fixed at One Support and Partly Fixed at the Other Support

Beams only partly fixed at one support depart somewhat from the condition of full continuity, and, therefore, may not yield to the application of the $\frac{1}{8}$-moment principle without small errors. Take, for example, an end-span slab having $M = \frac{1}{14} W l^2$ at mid-span, $M = \frac{1}{14} W l^2$ at the interior support, and $M = \frac{1}{28} W l^2$ at the exterior support. Referring to Fig. 3, let $B\,D\,B$ be the base line on which is drawn a parabola, $B\,C\,B$, representing the bending moment produced on a simple span beam by a load, W, per foot of span, giving a maximum moment, $M = \frac{1}{8} W l^2$. Draw the line, $B_4\,S_4$, in such a position that $H\,C = \frac{8}{14} D\,C$. The parabola, $C_4\,C\,C_4$, represents the bending moment between the points of contraflexure, $C_4\,C_4$. Then, $C_4\,C_4 = 0.756\ l$ and the reactions at C_4 from the load, W, are $0.378\ W\,l$. Let $Y\,L$ be the distance from C_4 to the support, S_4, Y being an unknown fraction. Then, assuming that the moment produced at the support, $S_4 = \frac{1}{14} W l^2$,

$$W l\ (Y l) + \frac{W (Y l)^2}{2} = \frac{1}{14} W l^2 = 0.071428\ W l^2$$

from which, $Y = 0.157$.

Similarly, assuming that the bending moment at the exterior support is $\frac{1}{28} W l^2$, the distance, $B_4 C_4$, is found to be equal to 0.085. As a check the distance, $C_4 C_4$, between the points of contraflexure is computed as,

$$1.000 - (0.157 + 0.085)\ l = 0.758\ l$$

showing an error of about one-half of 1 per cent.

Following the same procedure for a beam, $C_2 C_2$ (Fig. 3), having mid-span and end moments of $\frac{1}{11} W l^2$ and $\frac{1}{22} W l^2$, respectively, the corresponding dimensions on Fig. 3 are deduced. In this case a check on the distance between the points of contraflexure shows an error of less than one-third of 1 per cent.

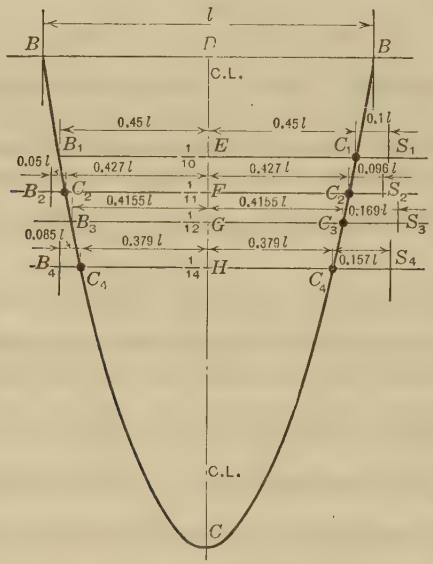

Fig. 3.

Beams Fixed at One End and Freely Supported at the Other End

Beams with one fixed end and one simply supported end depart radically from true monolithic construction; therefore, a greater error may be expected in applying the $\frac{1}{8}$-moment principle to them. For a beam, $B_1 E C_1$ (Fig. 3), for which $M = \frac{1}{10} W l^2$ at mid-span, $M = \frac{1}{20} W l^2$ at the interior support, and $M = 0$ (simply supported) at the exterior support, by a line of reasoning similar to the previous examples, the distance, $C_1 S_1$, is found to be equal to 0.1 l. Then the distance from C_1 to the free support, B_1, is equal to 0.9 l instead of 0.894 l, an error of slightly more than 1 per cent.

In like manner for the beam, $B_3\ G\ B_3$ (Fig. 3), in which the moments are $\frac{1}{12}\ W\ l^2$, $\frac{1}{12}\ W\ l^2$, and 0, computations show an error of slightly more than 3%, which is not enough to affect the design seriously; if accurate results are desired, however, this end span should be designed for $M = \frac{3}{35}\ W\ l^2$ at mid-span and $M = \frac{3}{35}\ W\ l^2$ at the interior support.

Spans of Unequal Length

The $\frac{1}{8}$-moment principle is useful in designing adjacent spans of unequal lengths. With two spans, freely supported at the exterior supports, design the shorter span for $M = \frac{1}{10}\ W\ l^2$ at mid-span and $M = \frac{1}{20}\ W\ l^2$ at the interior support. Compute the area of steel required in the longer span for $M = \frac{1}{8}\ W\ l^2$ and deduct from this one-half the area that the shorter span beam will develop in the top at the interior support.

For three spans, consisting of a short span in the middle and two longer end spans freely supported at the exterior supports, make the beam section of the short span the same size as for the longer end spans; design the end spans for $M = \frac{1}{10}\ W\ l^2$ at mid-span and $M = \frac{1}{20}\ W\ l^2$ at the interior support and carry the top steel the entire length of the short middle span. If conditions are reversed and there are two short end spans with a long interior span, design the end spans in the usual way and determine the area of steel for the interior span for $M = \frac{1}{8}\ W\ l^2$. Deduct from this the average area of top steel that the adjacent span beams will develop at the supports to give the area of steel in the center of the span. The area of steel can be checked from the relation that the area of steel in the bottom at mid-span plus the average of the areas in the top at the supports must be equal to the area required by the equation, $M = \frac{1}{8}\ W\ l^2$.

Depths of Concrete Beams

The designer has a wide range of choice in the size and shape of structural concrete members. There should be some definite principles or standards to assist him in the selection of the proper size and form, instead of the "random guess" method generally used.

The effective depths of concrete beams should be a definite ratio of the span length. Then, for a given load per linear foot of span, if the span length is doubled, the reaction at the support will be doubled, and if the width

of the beam is constant, the net area of the beam at the support will be doubled, and, consequently, the unit shear at the support will remain constant for all span lengths.

Referring to Fig. 1, let,

l = length of span, in feet.
W = load per linear foot of span.
b' = width of the stem, in inches.
d = effective depth of beam, in inches.
j = ratio of lever arm of resisting couple to depth, d.
V = total shear at support = $\frac{W\,l}{2}$.
x = ratio of d, in inches, to l, in feet.

The net area of the beam section, $b\ d = b\ x\ l$, and the maximum unit shear, v, at the support is:

$$v = \frac{V}{b'\,j\,a} = \frac{\frac{W\,l}{2}}{b'\,j\,x\,l} = \frac{W\,l}{2\,b'\,j\,x}$$

Notice that the factor, l, cancels out of the general equation, showing that the unit shear at the support is constant for all span lengths for a given load, W, provided b' and x remain the same. This principle makes it possible to draw simple shear diagrams which will show for any ratio, x, $\left(= \frac{d}{l}\right)$, the width of beam necessary to keep the unit shear at the support within any maximum specified. This diagram (Fig. 4) also will show at a glance the correct size of beam for any given load, W, and the unit shear produced at the support by that load for any span length. This principle would have an ideal application to the metric system of measurements. It gives satisfactory results with American units—feet and inches. The following ratios for x are recommended: $\frac{3}{4}$, $\frac{7}{8}$, 1, $1\frac{1}{8}$, $1\frac{1}{4}$, $1\frac{3}{8}$, $1\frac{1}{2}$, $1\frac{3}{4}$, and 2 in. per ft. of span. These will give ranges of depth from one-sixteenth (for $\frac{3}{4}$ in. per ft.), to one-sixth of the span (for 2 in. per ft. of span), or nine different depths of beam for any load and span length. Fig. 4 shows this shear diagram for a maximum unit shear in the concrete at the support of 120 lb. per sq. in.

For example, a concrete beam is to be designed with a clear span of 24 ft., carrying a total load of 2 800 lb. per lin. ft. Referring to Fig. 4, for an effective depth = $\frac{3}{4}$ in. per ft. of span, that is, for $x = \frac{3}{4}$, it is seen at a glance that a beam 16 in. wide and 20 in. deep is required to carry this load, and that the shear at the support would be about 117 lb. per sq. in. If d were taken as $\frac{7}{8}$ in. per ft. of span, the beam would be 14 in. wide and 23 in. deep and the shear at the support would be 114 lb. per sq. in.

Design of Shear Reinforcement

The determination of the size, number, and spacing of stirrups to properly care for the shearing stresses in concrete beams and girders is one of

the most tedious and time-consuming problems which the designer has to solve, and one for which no practical or rational method is presented in any of the technical literature on reinforced concrete designing. One textbook, published in 1923, states: "It is usually satisfactory to select a certain size of stirrup and calculate the spacing at different points along the beam." This involves a pure guess as to the number and spacing of the stirrups. Guesswork should not be countenanced in the field of design.

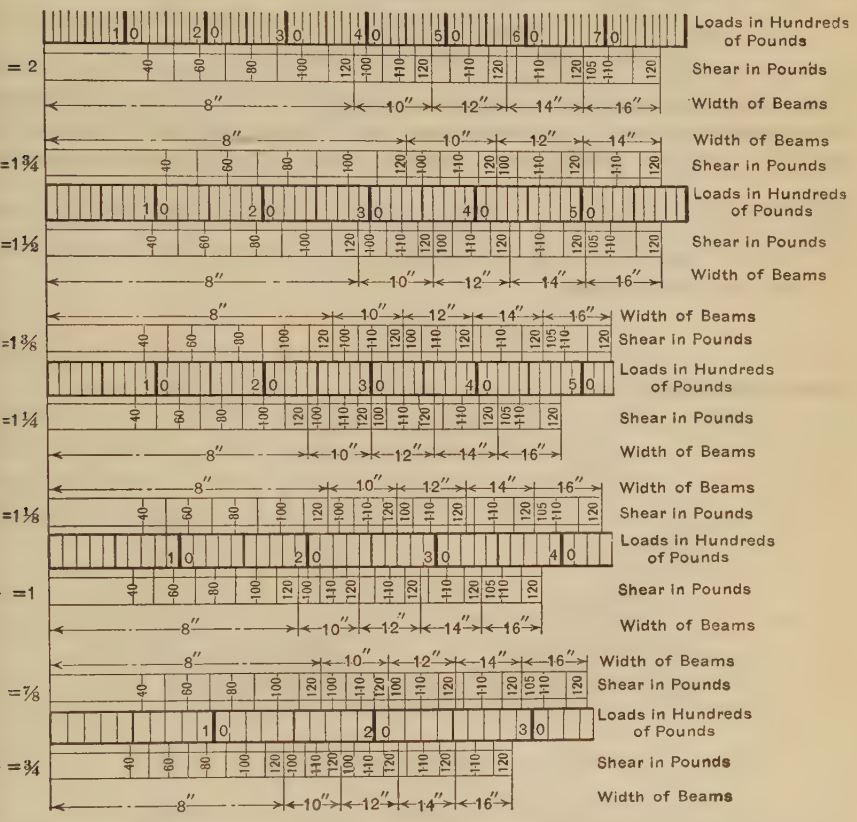

FIG. 4.—SHEARS CORRESPONDING TO DIFFERENT LENGTHS AND WIDTHS OF BEAMS FOR VARIOUS VALUES OF RATIO OF LENGTH, *l*, IN FEET, TO EFFECTIVE DEPTH, *d*, IN INCHES.

In this discussion the following standard symbols will be used:

s = spacing of stirrups, in inches.
N = total number of stirrups per beam.
A_v = area of the vertical members of a stirrup, in square inches.
A_s = total area of vertical shear steel, in square inches = $N\ A_v$.
f_s = stress on stirrups, in pounds per square inch.
j = ratio of lever arm of resisting couple to the effective depth of the beam = 0.875 average.
d = effective depth of beam, in inches.

b' = width of stem of T-beam, in inches.
V = total shear on Section $b'\ d$, in pounds per square inch.
v_1 = unit shear on Section $b'\ d$ carried by stirrups, in pounds per square inch.
v_2 = allowable unit shear on plain concrete = 40 lb. per sq. in.
V_1 = total shear carried by stirrups = $v_1\ b'\ d$.
V_2 = total shear carried by plain concrete = $v_2\ b'\ d$.
l = clear span of beam, in inches.
l' = distance, in inches, from support to the point where the total shear on Section $b'\ d$ = 40 lb. per sq. in.

In Fig. 5, the triangle, $A\ B\ C$, represents the shear diagram for one-half a beam carrying a uniformly distributed load. The area, $D\ B\ C\ E$, represents the shear carried by plain concrete and the area, $A\ D\ E$, the shear carried by the stirrups. Note that $\frac{V_1\ l'}{2}$ is the area of the diagram representing the shear carried by stirrups for one end of the beam.

From similar triangles:

$$l' : V_1 :: \frac{l}{2} : V \qquad \ldots\ldots(1)$$

From Equation (1),

$$l' = \frac{V_1}{2\ V} \times l = \frac{v_1\ l}{2\ V} \qquad \ldots\ldots(2)$$

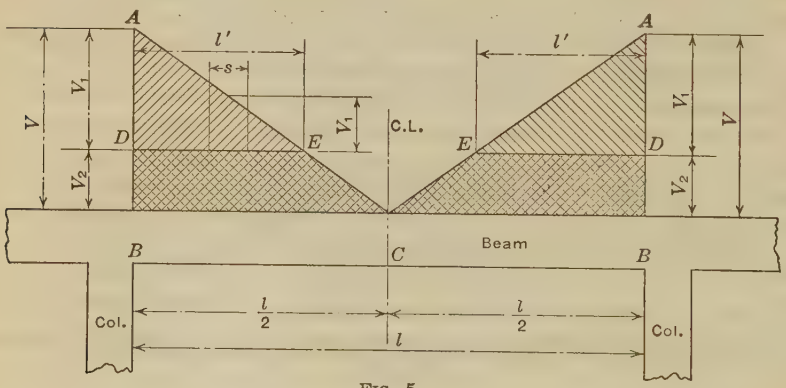

FIG. 5.

The equation generally used for determining the spacing of stirrups is,

$$s = \frac{A_v f_s\ j\ d}{V_1} \qquad \ldots\ldots(3)$$

Equation (3) may take the form,

$$V_1\ s = A_v f_s\ j\ d$$

$V_1\ s$ represents the area of the shear diagram carried by one stirrup; but $V_1\ s = A_v f_s\ j\ d$, therefore, $A_v f_s\ j\ d$ may be taken as the area of the shear

diagram carried by one stirrup. Consequently, the total number of stirrups required per beam is,

$$N = \frac{V_1 \, l'}{A_v f_s j \, d} \quad \ldots\ldots (4)$$

but,

$$V_1 = v_1 \, b' \, d$$

therefore,

$$N = \frac{v_1 \, b' \, d \, l'}{A_v f_s j \, d} = \frac{v_1 \, b' \, l'}{A_v f_s j} \quad \ldots\ldots (5)$$

Note that d disappears from Equation (5) and that, consequently, for any given value of v_1, the number of stirrups required is independent of the depth of the beam. The depth of the beam influences the number, however, by its effect on the value of v_1.

In applying Equation (5) it is necessary to assume the size of stirrups to be used in order to ascertain the value of A_v. Equation (5) may be transformed to give the total area of stirrups as follows:

$$A_s = N \, A_v = \frac{v_1 \, b' \, l'}{f_s j} \quad \ldots\ldots (6)$$

from which A_s or N may be found.

To illustrate the application of these equations, assume a beam with a clear span of 20 ft., 14 in. wide, and 20 in. deep, carrying a total uniform load of 3 000 lb. per lin. ft. The reaction at each support is 30 000 lb.; the shear per square inch on Section $b' \, d$ at the support is 119 lb. Now, applying Equation (2):

$$l' = \frac{v_1 \, l}{2 \, V} = 80 \text{ in.}$$

Applying Equation (5), and assuming $\frac{1}{2}$-in. round stirrups:

$$N = \frac{v_1 \, b' \, l'}{A_v f_s j} = \frac{79 \times 14 \times 80}{0.39 \times 16\,000 \times 0.875} = \text{sixteen } \tfrac{1}{2}\text{-in. round stirrups}$$

Or, by applying Equation (6),

$$A_s = \frac{v_1 \, b' \, l'}{f_s j} = \frac{79 \times 14 \times 80}{16\,000 \times 0.875} = 6.32 \text{ sq. in.}$$

Spacing of Stirrups

Except in rare cases, the bent longitudinal bars should not be considered as shear reinforcement. They vary much in size, in their position in the beam with respect to the support, and in the angle of the bent portion. If the plain concrete can carry, say, 40 lb. per sq. in. at the support, it can carry 40 lb. throughout the entire length of the beam. The assumption that the concrete may carry one-third of the shearing stresses at any point in the beam is supported by no logical basis. The plain concrete is usually considered to take a shearing stress of 40 lb. per sq. in. on the section, $b \, d$, and stirrups are used to care for the additional shearing stress. They should be

spaced so that they are subjected to a theoretically uniform stress. Each stirrup should care for its proportional area of the shear diagram.

The area, *A D E* (Fig. 6), represents the shear carried by the stirrups at one end of the beam of Fig. 5. Theoretically, using sixteen ½-in. round stirrups in the beam, eight at each end, this triangle, *A D E*, should be divided into eight equal areas and a stirrup placed at the center of gravity of each area; but, if *A D E* is divided into sixteen equal areas, stirrups placed at each odd numbered line of division will practically coincide with the eight gravity lines mentioned. Since one of the functions of a stirrup is to give a vertical support to the beam bars, there should be an extra half space adjacent to the support, giving $N + 1$ spaces in all.

This spacing may be accomplished by the following analytical method. From plane geometry the areas of similar right triangles are proportional to the squares of the respective sides, and, conversely, the sides of similar right triangles are proportional to the square roots of the respective areas. Therefore, referring to Fig. 6,

$$y_1 : l' :: \sqrt{1} : \sqrt{N+1}, \text{ or, } y_1 = \frac{80}{\sqrt{17}} = 19.4 \text{ in.}$$

In the same manner:

$$y_1 : y_3 :: \sqrt{1} : \sqrt{3}, \text{ or } y_3 = 33.6 \text{ in., etc.}$$

All such computations giving the spacings shown on Fig. 6 may be accomplished by a single setting of the slide-rule.

If the slide-rule is read to the nearest inch, no stirrup will be more than ½ in. from its theoretical position, and this will be much more accurate than the actual location.

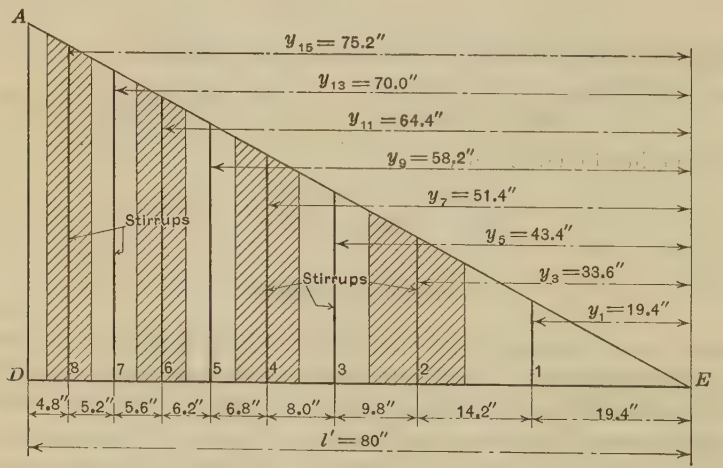

FIG. 6.

It should be noted that in all cases of fixed beams, the part of the beam from the support to the point of contraflexure acts as a cantilever beam and, consequently, its stirrups should be inverted. In spans with one end freely

supported and the other end fixed, the reactions are, respectively, 0.55 $W\ l$ and 0.45 $W\ l$. The unit shears are determined in the usual manner; the stirrups at a free end are the same as those for a simple span the length of which is 0.9 l, and the stirrups at the interior support are the same as those in an interior span the length of which is 1.1 l.

It is hoped that the foregoing ideas on arranging longitudinal and shear reinforcement will clarify this general question or at least stimulate thought and expression of opinion from which some advance will result. If this one point of view provokes discussion tending to rationalize the design of concrete beams and slabs, it will have served a useful purpose. The moment coefficients and design diagrams represent ideas on reinforcement that have proved extremely serviceable to the writer.

DISCUSSION

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—The portions of this paper that the writer would like to discuss concern the bending moments in flat slabs and the stirrup as a shear member.

Mr. Myers has shown that, taken as a standard or basis for computing bending moments, the value, $\frac{w\,l^2}{8}$, or the bending moment on a simple beam, has a peculiar place for different conditions of continuity or fixity of supports. On page 178, he states:

"In all cases the sum of the center moment and average moment at the supports is equivalent to the moment for a simple beam of the same span."

While this is strictly and undeniably true, as can be readily demonstrated, there is not a standard for designing flat slabs that makes use of this principle or even approximates the reinforcement in the slab that this principle clearly demonstrates to be necessary.

Considering a flat slab as supported on lines of parallel columns and applying the principle just quoted, it is seen at once that the bending moment on a mid-section of the slab parallel with the lines of columns, added (numerically) to the bending moment on a section through the column centers, should be equal to one-eighth of the total load carried between the two rows of columns times the span between columns. No method of designing flat slabs provides reinforcement that even approximates this requirement.

If the lines of support of the flat slab are considered to be tangent lines touching the circles of the column heads, the case is simpler and the column heads and columns do not complicate the calculations. There is no code in use nor standard of design that provides reinforcement even approaching the requirements of the case as quoted.

There is no reason why flat slabs should be given this advantage over other slab designs. It is true that some tests on flat slab structures have appeared to show a strength that justifies the present standards. This is because those who have made these tests have never been willing to make a critical test on flat slab construction.

Single bays, or several selected bays, when tested in a flat slab floor, show great strength because the surrounding, idle floor aids the parts that are loaded. Spot loads that produce a dished shape in the floor slab do not test the floor critically. The critical load on a flat slab floor is a load that covers a row of bays clear across a building from wall to wall. A load of this kind deflects the floor in a cylindrical instead of a dished shape. The bending moment in the slab is quite definite under this loading, and it cannot be otherwise than that stated by Mr. Myers.

The flat slab problem is attacked from the standpoint of complex mathematics involving such factors as Poisson's ratio, and yet the simple criterion expressed in the paper is a definite and determining criterion which cannot

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be gainsaid. It reduces investigation of the flat slab, from the mathematical standpoint, to the simplest terms.

The other portion of the paper to which the writer would call attention is that part where the spacing of stirrups is derived. It is a *sine qua non* of any attempt to discuss the spacing of stirrups or to derive rules for that spacing, that one must first justify the stirrup and the basis of the method by which the rule of spacing is derived.

Mr. Myers mentions the paucity of rational methods of spacing stirrups in the literature of reinforced concrete. The real lack is a rational analysis of the stirrup itself. All treatment of stirrups in standard literature starts with the hypothesis that in some way, which it is not necessary to defend or explain, a stirrup will take an amount of shear corresponding to the vertical shear in the beam.

From the standpoint of analysis: It is many years since any authority has held that stirrups perform their function by resisting shearing stress in the steel itself, so that the idea of stirrups taking horizontal shear and transmitting this as increments into the horizontal rods has been abandoned. Shear in the steel rods is therefore eliminated from the analysis.

If the stirrup takes vertical shear as a compression stress, it would mean that the beam is shortened in its height from top to bottom. It is conceivable, if a layer of soft substance existed in a beam, in a diagonal direction, crossing a stirrup and sloping downward from the top of the beam toward the middle of the span, that compressive stress of considerable magnitude would be developed in that stirrup. A trifling amount of compressive stress could develop in stirrups due to the sag or deflection of fixed ended beams; but no authority mentions compression as the kind of stress for which stirrups are to be designed.

The stirrup must be considered, therefore, as a tension member. Regarding its action there are two types of analysis, both of which are merely fragmentary. One analysis shows a 45° section cutting the beam and the stirrups. The shear would be a tension on this diagonal section, and the analysis would divide this tension by the number of the stirrups cut to obtain the tension in one stirrup. This analysis ignores the fact that the section may cut through the very tip end of one or two of those stirrups, where the stirrup could have no anchorage or hold in the concrete. If the beam were actually severed in this plane, these stirrups without anchorage would be useless, and yet they are given the same weight and value as stirrups that have some, if not a full, anchorage. In fact, few stirrups have anchorage enough to take the tension allotted to them in the design. To gain some idea as to the length needed for anchorage, suppose the stirrups are $\frac{3}{8}$-in. square bars, and assume 40 diameters as the length needed for anchorage. The stirrup would require 30 in. of length for anchorage alone and only the middle of the length would be properly conditioned for its stress. It will be readily appreciated that stirrup and beam sizes do not conform to the dimensions that analysis thus shows to be necessary.

It will be said, however, that anchorage for stirrups is effected by looping and hooks rather than by embedment in the concrete. It has been demonstrated by tests that an ordinary loop does not constitute a proper and

full anchorage for a steel rod. To assume that a loop of the stirrup effects this anchorage is to presuppose that the shear load is hung as a vertical load on the horizontal rod. The horizontal rod is not capable of carrying concentrated loads such as this.

It is these features of stirrup analyses that are ignored by authorities and that render the analyses of no value, since they do not follow the stresses out to adequate resistance.

The other analysis to which reference has been made is summed up in vague statements that stirrups act as the web members of a truss. Here, too, no effort is made to follow out these alleged web stresses to adequate resistance —to show how the stress is imparted to the stirrup and to what that stress is delivered after it passes through the stirrup. In truss analyses these matters are taken care of to the last pound in adequate details and sections; in reinforced concrete analyses they are totally ignored. What forces could there be in a beam stretching it out in a vertical direction to give tension in the stirrups?

From the standpoint of tests: No tests that the writer could ever find have shown anything but a trifling amount of compression in stirrups in a whole beam. Beams with diagonal shear cracks have exhibited high tension in the stirrups crossing these shear cracks; nothing else could be expected; but why did not the stirrups prevent the shear cracks, if they were performing their function? And why, in the whole beams, did the stirrups not show tension of considerable amounts rather than compression in trifling amounts, since it is tension for which they are designed? The confirmation of analysis is tests that at least approximate agreement with that analysis.

There is a space between stirrups where the full shear of the beam must be carried by the concrete. Between the last stirrup and the support this shear is the maximum, but there is no reinforcement in the beam of the standard design referred to by Mr. Myers to relieve the concrete of shear in this critical section. He specifically rejects reinforcement that would satisfy the requirement of steel reinforcement for all vertical sections under high shear, in these words, "Except in rare cases, the bent longitudinal bars should not be considered as shear reinforcement."

It is astonishing that Mr. Myers should reject the only shear reinforcement that is capable of any kind of analysis, which is at the same time a type of reinforcement that positively prevents a beam from failing. A beam that depends on stirrups or any kind of short shear reinforcement to take its shear may fail completely in shear by severing the concrete between the stirrups, without in any manner disturbing the stirrups. Hundreds of beams in buildings which collapsed have failed in exactly this manner. A beam in which the main reinforcing rods are bent up and anchored beyond the edge of support cannot possibly fail without either severing the rod completely or pulling it out of the concrete in which it is fully anchored. There could be no more positive reinforcement than this—and this is the type of reinforcement which Mr. Myers says should not be considered, except on rare occasions.

Many designers are using this type of reinforcement for shear, and it is safe to say that there have been no cases of beams that have fallen away from their supports where this type of reinforcement has been used.

The analysis of the stress in bent-up and anchored rods is quite simple. It is the old principle of the hog-rod or the queen-post truss. The shear of the beam for which reinforcement is needed acts as a vertical force, and the component in the direction of the rod is the stress in that rod for which its section and anchorage should be proportioned.

The rods should not be bent down at different distances from the support, but the downward bend should be about at the edge of the support. The upward bend should be where the shear is small enough to be taken by the concrete at a unit stress of 50 to 60 lb. per sq. in. If there are concentrated loads, as of beams carried by a girder, the bend of the rod should be under the girder.

Steep bends in main rods and bends at different distances from the support cannot be defended or analyzed. The rods should be bent up in one plane if more than one rod is needed for the shear, and the slope of the rod should be a flat one. This avoids sharp bends; for no reinforcing rods should be under high stress at the bends, if these bends are sharp.

The foregoing rules for designing shear reinforcement would supplant all rules for the spacing of stirrups; they are based on a rational and complete analysis of the beam stresses. The system leaves to the concrete the mere function of holding by proper embedment and anchorage the reinforcing steel, and places on the steel the function of sustaining direct, calculable tension, which both the concrete and the steel are capable of and fitted to do.

It is significant to note that the same series of tests, made on existing buildings, which disclosed small compressive stresses in stirrups in beams that were cracked, showed large stress in shear reinforcement consisting of bent-up and anchored main steel rods, the type of shear reinforcement that Mr. Myers would reject; and no cracks could be found in the beams thus reinforced.

Mr. Myers' requirement that the steel reinforcement be designed to take all the shear is an unnecessary burden on the design. Concrete is abundantly able to take shear up to amounts of 50 to 60 lb. per sq. in. provided it is safeguarded against the start of a failure produced by a crack. If the horizontal reinforcement extends well beyond the edge of support that reinforcement will tie the concrete together on the tension side of the beam and render it capable of carrying the unit shears just named with complete safety.

H. C. Sandbeck,* Jun. Am. Soc. C. E. (by letter).—It is a general principle that in the ideal design of a structure the same factor of safety applies throughout. For the sake of simplicity, however, the engineer often is obliged to build certain parts with excess strength. He either "figures on the safe side" to simplify calculation, or he orders superfluous material to simplify erection; but he will never for the sake of simplicity allow a single part of a structure to have a smaller factor of safety than is required for the structure as a whole.

* Care, Central Hudson Gas & Elec. Corporation, Poughkeepsie, N. Y.

It seems, however, that some of the simplifications recommended by Mr. Myers in calculating reinforcing steel for concrete beams and slabs will lead to designs which have a considerable smaller factor of safety than is generally required for reinforced concrete.

The writer agrees with the "General Principles" advanced by the author that reinforced concrete beams and slabs are non-homogeneous and have a varying moment of inertia. (It is believed, however, that this latter fact can be taken into consideration in the proper use of the elastic theory with only slight error.) Further, it is correct in a majority of cases to consider monolithic built beams and slabs as fixed at their supports. These considerations, however, will not justify a reduction of steel over supports such as Mr. Myers recommends.

For beams and slabs continuous for more than two spans the Joint Committee on Specifications for Concrete and Reinforced Concrete specifies* that for maximum positive moment near the center and maximum negative moment at the support of interior spans, $M = \frac{1}{12} w l^2$. Mr. Myers states that these beams "should be designed for $M = \frac{1}{12} w l^2$ at mid-span and $M = \frac{1}{24} w l^2$ at the supports."

It is well known that a uniformly loaded beam fixed at both ends and with a constant moment of inertia, produces a bending moment, $M = + \frac{1}{24} w l^2$ at the center, and $M = - \frac{1}{12} w l^2$ at the supports. Assuming the moment of inertia to have a constant value, I_N, where the bending moment is negative and another constant value, I_P, where the bending moment is positive, it is an easy matter to calculate the moments produced.

This assumption is not quite correct because concrete in tension will add to the stiffness of the beam where it is not cracked (near points of contraflexure); but as the steel is dominant in determining the stiffness, the approximation is believed to be very close.

A uniformly loaded beam, fixed at the supports, has a moment diagram, as shown in Fig. 7 (*a*), and an elastic line, as shown in Fig. 7 (*b*). Any two tangents to the elastic line intersect at an angle, ϕ, such that,

$$\phi = \int \frac{M}{E I} d x$$

the integral being taken between the tangent points.

On account of symmetry, the tangent to the elastic line at B' is parallel to the tangent at C', therefore, between B and C:

$$\int \frac{M}{E I} d x = 0$$

As $\int M \, d x$ is the area of the moment diagram, the positive area, B-B''-D, divided by I_P must equal the negative area, D-C''-F, divided by I_N.

* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1190.

The formula for the parabola in a co-ordinate system, $X = Y$, is $y = k\,x^2$. For $x = \frac{l}{2}$, y is $\frac{w\,l^2}{8}$, and as $\frac{w\,l^2}{8} = k\,\frac{l^2}{4}$, therefore, $k = \frac{w}{2}$, and, generally, $y = \frac{w}{2}\,x^2$.

The area, *B-B″-D*, is:

$$\frac{2}{3}\,a\,.\,b = \frac{2}{3}\,a\,.\,\frac{w}{2}\,a^2 = \frac{w}{2}\,.\,\frac{2}{3}\,a^3$$

The area, *D-C″-F*, is:

$$\int_a^{\frac{l}{2}} y\,d\,x - b\left(\frac{l}{2} - a\right) = \int_a^{\frac{l}{2}} \frac{w}{2}\,x^2\,d\,x - \frac{w}{2}\,a^2\left(\frac{l}{2} - a\right)$$

$$= \frac{w}{2}\left[\frac{l^3}{24} + \frac{2}{3}\,a^3 - \frac{a^2\,l}{2}\right]$$

Then,

$$\frac{1}{I_N}\,.\left[\frac{l^3}{24} + \frac{2}{3}\,a^3 - \frac{a^2\,l}{2}\right] = \frac{1}{I_P}\,\frac{2}{3}\,a^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

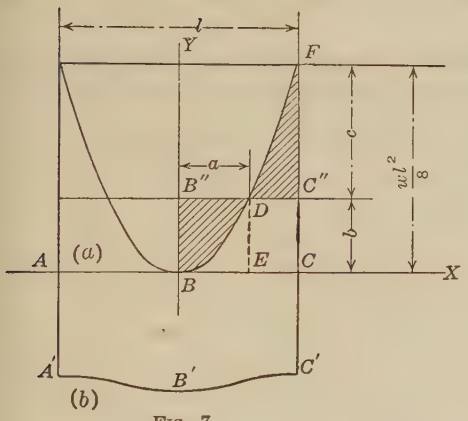

Fig. 7.

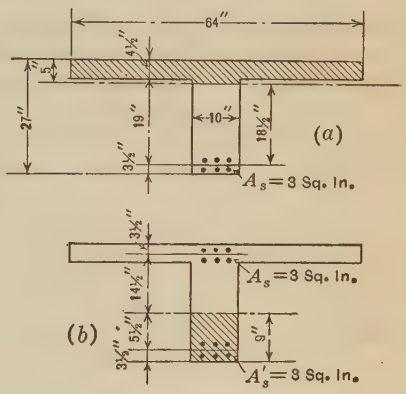

Fig. 8.

From Equation (7), a, and, therefore, b, can be found when I_P and I_N, or the proportion between them, are known.

It will be seen that if $I_P = I_N$, a^3 cancels, and Equation (7) will be written,

$$a^2 = \frac{1}{12}\,l^2$$

therefore,

$$b = \frac{w}{2}\,a^2 = \frac{1}{24}\,w\,l^2$$

as is given in all handbooks.

In order to get an idea about how much the moment of inertia changes at the points of contraflexure, consider the beam shown in Fig. 8. This beam is designed for $M = \frac{w\,l^2}{12} = 1\,000\,000$ in-lb. at both the center and the sup-

port. One-half the positive reinforcement is bent up over the supports, and all bars overlap at the supports. The cross-section at the center is shown in Fig. 8 (*a*) and at the supports in Fig. 8 (*b*). Neglecting tension in the concrete, the moment of inertia can be computed, as follows:

$$I_P = \frac{1}{12} \times 64 \times 4.5^3 + 64 \times 4.5 \times 2.75^2 + 15 \times 3 \times 18.5^2 = 18\,120 \text{ in.}^4$$

$$I_N = \frac{1}{3} \times 10 \times 9^3 + 15 \times 3\,(5.5^2 + 14.5^2) = 13\,230 \text{ in.}^4$$

$$\frac{I_N}{I_P} = \frac{13\,230}{18\,120} = 0.73$$

If this value is used in Equation (7), the value of a will be found (by trial) to be $a = 0.3l$.

Therefore,

$$b = \frac{w}{2} a^2 = 0.045\, w\, l^2 = \frac{1}{22.2} w\, l^2$$

$$c = 0.125\, w\, l^2 - 0.045\, w\, l^2 = 0.08\, w\, l^2 = \frac{1}{12.5} w\, l^2$$

The design has nearly the desired safety factor over the supports and is about 36% too strong at the center.

If A_s and A_s' in Fig. 8 (*b*) were reduced to 1.5 sq. in., the neutral axis would move to 7.5 in. from the bottom, and the moment of inertia would be:

$$I_N = \frac{1}{3} \times 10 \times 7.5^3 + 15 \times 1.5 \times (7.5^2 + 16^2) = 8\,500$$

$$\frac{I_N}{I_P} = \frac{8\,500}{18\,120} = 0.468$$

Computed as before, the moments will be,

$$M = + \frac{1}{18.2} w\, l^2$$

and

$$M = - \frac{1}{14.3} w\, l^2$$

in the middle and over the support, respectively.

The section has a resisting moment:

$$M = f_s \times p \times j \,.\, b\, d^2$$

$$= 16\,000 \times 0.0064 \times 0.90 \times 10 \times 23.5^2 = 500\,000 \text{ in-lb.}$$

The moment previously used was:

$$M = \frac{w\, l^2}{12} = 1\,000\,000 \text{ in-lb.}$$

therefore, this section is good for only one-half as much, $\frac{w\, l^2}{24}$, or just the amount recommended by Mr. Myers. The steel over the supports, however, would be strained to,

$$16\,000 \times \frac{24}{14.3} = 27\,000 \text{ lb. per sq. in.}$$

It seems inadvisable, therefore, to use less than the amount of steel over the supports recommended by the Joint Committee. By decreasing this amount, the negative bending moment is reduced; but the resisting moment of the beam is reduced still more, and the steel is over-stressed.

JAMES B. NEWMAN,* M. AM. SOC. C. E. (by letter).—This paper reveals much of interest and value, although the writer differs as to some points. He feels that the moments at the center and the supports are more the uncertain factors than the distribution of steel which necessarily follows, and that this problem with its limitations has been quite fully treated in current literature and texts. The following brief criticisms, therefore, will not be fully developed, references being furnished to more complete discussions.

The author has used many computations in showing for several numerical cases that the sum of the center moment and the average of the negative end moments of a beam carrying a uniformly distributed load of W per lin. ft. is $\frac{W L^2}{8}$. It is clear, by referring to Fig. 9 (a), that a beam, $A B$, carrying a uniformly distributed load of W per ft., and terminal negative moments, which may or may not be equal, is exposed to positive and negative moments as shown throughout the span. The resultant moment at any point, the algebraic sum of the positive and negative moments, is shown with respect to the base line, OO, Fig. 9 (b). It is clear from Fig. 9 (b), as pointed out in most texts, that the sum of the center positive ordinate and the average of the end terminal moments is $\frac{W L^2}{8}$. It should be emphasized, however, that this diagram only provides for a single position of the load, whereas even in building work a considerable portion of the live load may be moving, and while the complicated criteria for maximum shears and moments which hold in bridgework are out of place here, some reasonable consideration may be given to varying positions of the live load.

Theoretical considerations of continuous beams resting on simple supports† and of continuous beams in monolithic constructions‡ show that, for varying ratios of live and dead load and varying positions of the live load in successive or intermittent spans, the positive or the negative moment may cover practically the entire span. Moreover, the maximum value that the negative moment may readily attain, plus the maximum value that the positive moment may attain, considerably exceeds $\frac{W L^2}{8}$. In fact, very reasonable values which may be attained as to magnitude and distribution are shown in Fig. 10. It is true of course that such positive and negative moments could not exist simultaneously, but as either the positive or negative moments here shown can be attained singly, the design should be suitable to provide

* With Buchman & Kahn, New York, N. Y.

† "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, and other texts.

‡ Concrete Engineer's Handbook, by Hool and Johnson.

for both. In this case, therefore, the total steel required would be based on $\frac{W L^2}{6}$.

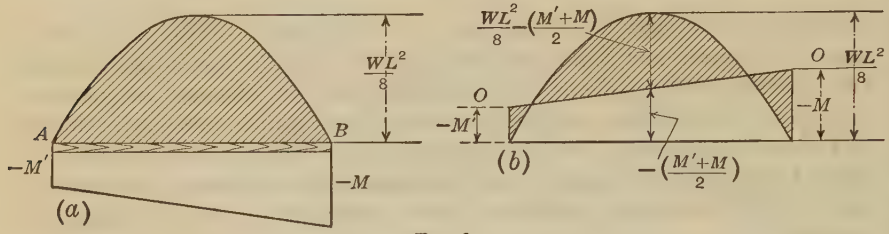

Fig. 9.

The author assumes that the moment transmitted to the column is a direct function of the negative steel in the beam at the support instead of being simply limited by that steel. The monolithic structure is statically indeterminate to be sure, and as a practical expedient, a certain moment may arbitrarily be assigned to the beam at the ends, obtaining the center positive moment by subtracting the assumed end moment from a reasonably determined total moment, and then designing the structure to adequately care for these moments. The design will then be consistent, and safe, but this does not mean that it will be either rational or good. Some parts may be seriously over-stressed, as the author pointed out in the case of the monolithic beam with all positive steel.

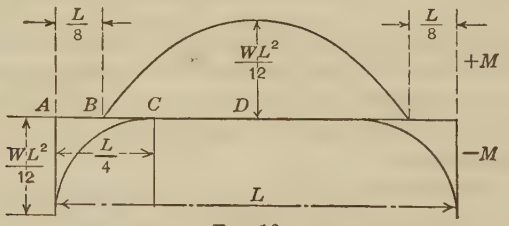

Fig. 10.

A steel bent carrying a single transverse load, for example, may be so designed that the member, *DE*, Fig. 11 (*a*), carries the entire load. However, distortion of this member will throw stress into the member, *FG*. The bent may be designed as in Fig. 11 (*b*), with *EF* and *FG* carrying the load, but then under distortion, *DE* will be over-stressed. It may be assumed that *DE* and *FG* carry the stress equally, Fig. 11 (*c*), although due to shortening of *EF* it is not true. While all these structures will stand if consistently designed for these conditions, all are not equally rational. The actual determination of the moments at the supports and the center of the beam is a matter of mechanics and not of assumptions. It is only because some of the practical factors which enter into the situation are unknown, and must be appraised that there is any justification for arbitrary assignment, but assump-

tions should approximate the true values, as nearly as they can be reasonably worked out, from consideration of statical laws.

The moment the beam carries at the support must be adequately provided for, however troublesome that may be. As the author suggests, some bottom steel may be used, although this is expensive, and the section may be designed as a doubly reinforced beam; additional depth may be gained by haunching; or as is usually the case, this section may be designed as a rectangular beam, thus determining the width and depth of the stem throughout. This may give a larger concrete section than is actually required at the center, based on the T-section. The stresses at the support are often taken a little higher than at the center, on the theory that the maximum stress falls off much more rapidly than in the center where the moment curve is relatively much flatter; also because of the column effect, both in providing a relatively wide support, and, by construction, tending to reduce moments theoretically deduced. It should be mentioned, however, that this column effect has already been partly discounted in the assumption of moment factors shown in Fig. 10.

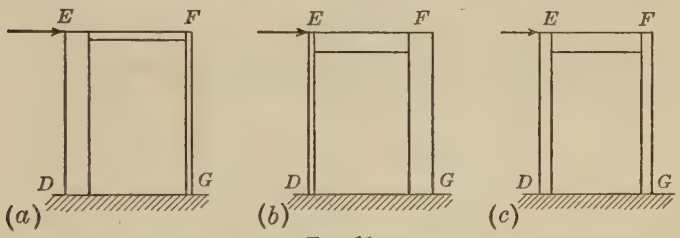

FIG. 11.

The author's discussion of shearing stresses and stirrups is along the generally accepted lines and requires little comment. There is ground for some difference of opinion here too, however. The stress that the stirrups are assumed to carry is the vertical component of the tensile principal stress. The longitudinal or transverse shear as ordinarily computed is used as a gauge of this diagonal tensile stress and, therefore, it is said in common parlance that the stirrups carry the shear not carried by the concrete. The concrete beam as noted is non-homogeneous, and the actual principal stresses set up where there is considerable shearing stress and where the steel is assumed to carry all the tensile stress due to bending (although the concrete carries some, depending on the amount it has been cracked) cannot be known with any great degree of accuracy. It is known, however, that if the stirrup steel inserted is ample, the beam distortion is kept to a minimum and the beam acts more as if homogeneous. This shear reinforcement, for the good it does, is relatively inexpensive, and there is accordingly some justification for not skimping material. This is one justification for using the concrete to carry only one-third the shearing stress, to the point at which the shear is 40 lb. per sq. in., instead of a straight 40 lb. throughout.

Moreover, as in the case of the moments already considered, the author assumes the entire load to be static. A reasonable variation in shear may be briefly noted, however. A simple beam carrying a uniform load, W, per ft.,

carries a total end shear of $\frac{W}{2}$, and a center shear of O, the variation being as shown by the line, AB (Fig. 12). If the beam is loaded with the same intensity from one support to the center only, the end shear at the loaded support is $\frac{3\,W\,L}{8}$, and at the center, $\frac{W\,L}{8}$, this variation being shown by the line, DEF (Fig. 12). While both conditions cannot exist simultaneously, some designers assume a variation in shear following the line, AE, which provides for each case. It may be observed that using this latter distribution of shear and assuming 40 lb. as the concrete capacity throughout will tend to approach a shear variation from A to B, with the concrete carrying only one-third the stress to a point at which the unit shear is 40 lb.

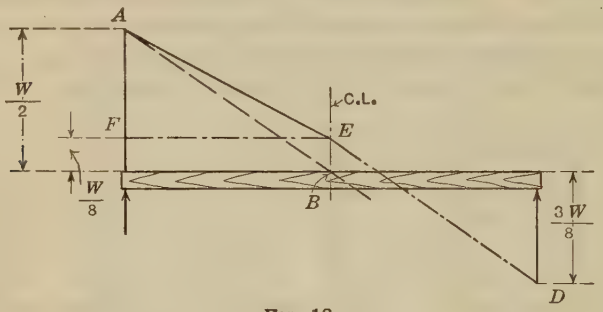

FIG. 12.

The convenient method of determining stirrup spacing used by the author has also been suggested by Turneaure and Maurer in their text, and a graphical division of the shear triangle into the several parts has been suggested by Hool and others.

It should be pointed out that any methods of determining stirrup spacing are not so easily applied nor so simply carried out when the shear curves show discontinuous and irregular variation.

E. F. ROCKWOOD,* M. AM. SOC. C. E. (by letter).—The author is not presenting anything new, as his recommendations were discussed and discarded by leading engineers twenty years ago. Herewith is a plot (Fig. 13), prepared by the writer at that time, to show one very serious error in Mr. Myers' reasoning.

It was known and admitted then that the moment at one end of a fixed beam plus the moment at the center equals the moment at the center of a simple supported beam of the same span and load conditions. Unfortunately, however, the distribution of moments between these two points is determined by the flexure of the beam and not by the whim of the designer.

If the beam is of uniform section throughout, the end moment is $\frac{W\,L}{12}$ and the center moment is $\frac{W\,L}{24}$. Mr. Myers recommends $\frac{W\,L}{12}$ at the center and

* Boston, Mass.

$\frac{W\,L}{24}$ at the supports. Therefore, it may be instructive to assume a beam designed in accordance with his recommendations and check it.

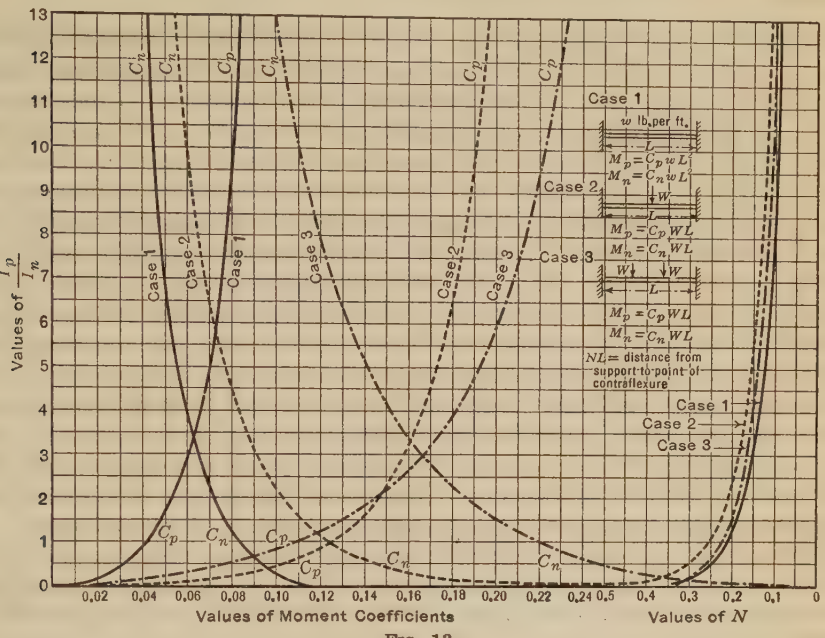

FIG. 13.

Let the ends of the beam have a section of 10 by 20 in., with 1% of steel. Stresses of 16 000 and 750 lb. per sq. in. are allowable. If $n = 15$, the negative moment of inertia, I_n, is $0.075\,bd^3 = 6\,000$. At the center assume the same stem and an 8-in. slab. The corresponding flange width is 138 in. and the percentage of steel is $\frac{4\,100}{138 \times 20} = 0.145$. The positive moment of inertia then becomes $0.01 \times 138 \times 20^3 = 11\,040$. Referring to Fig. 13, for a ratio of $\frac{I_p}{I_n} = \frac{11\,040}{6\,000}$, or 1.84, M_n becomes $0.05\ W\,L$ and M_p becomes $0.075\ W\,L$. Mr. Myers provides negative reinforcement corresponding to only $0.042\ W\,L$, and if the slab is thinner, I_p and M_p will decrease and M_n will increase until it becomes $0.083\ WL$ when $I_p = I_n$.

Many engineers would like to make the total steel in a concrete beam equal to $\frac{W\,L}{8}$ although they might, for the reasons herein stated, use a distribution different from that of Mr. Myers. However, except for beams of relatively small section compared with the columns into which they frame, most beams cannot be considered as fixed at the ends and if the ends are only partly fixed, the loadings of adjacent spans must be considered and the beam designed for positive moment under the loading that will give the largest probable nega-

tive moment. As these two moments will total more than $\frac{WL}{8}$, it is unsafe to design for a total moment of that amount except under conditions of fixed loading.

C. A. Willson,* Assoc. M. Am. Soc. C. E. (by letter).—Strictly this is not a discussion of the author's paper, but it is presented in accordance with his suggestion that other engineers contribute their own ideas on the subject.

In the structural design of buildings for the University of Wisconsin, the State Normal Schools, and the other State institutions, the writer has had frequent occasion to design continuous concrete floor-joists and floor-beams consisting of two or more spans of unequal length. For example, in the new Men's Dormitory at the University, the plan called for an inside width of building of 34 ft. 6 in., giving two rows of rooms at 14 ft. 0 in. each, separated by a corridor 6 ft. 6 in. wide. It did not seem economical to place a concrete beam and a row of columns lengthwise of the building on each side of the corridor, therefore only one beam and one row of columns was used. As a result, the concrete floor-joists had to be designed as continuous beams of two unequal spans, one 20 ft. 6 in. and the other 14 ft. 0 in. in length. At the new Nurses' Dormitory at the University the window treatment required an unequal spacing of wall columns. This meant that the spandrel beams had to be designed as continuous beams consisting of a series of unequal spans.

Moments at Supports.—Cases like this have been so numerous that the writer has found it necessary to develop methods for determining the moments in continuous beams of unequal spans with the maximum of speed and a relatively high degree of accuracy. In the design of continuous beams of unequal spans, the major problem is to determine the moments at the supports—the remainder of the design is comparatively simple.

Quite often in building construction, the maximum load per linear foot on a continuous joist or beam is the same for all spans. Theoretically, for a maximum moment at any support the two adjacent spans and each alternate span should be loaded.†

However, the load in the second span from any particular column has only a small effect on the moment at that column and any ordinary load on the third span will have practically no effect. Consequently, the writer uses a constant value for the load per foot for all spans in ordinary problems because the work is simplified thereby with only a slight sacrifice in accuracy.

Influence lines show that an excellent notion of the negative moment at any support of a continuous beam composed of a long series of spans may be obtained by considering only four spans, two on each side of the support in question. Later in this discussion a method of finding moment based on this idea of four spans will be developed.

The two spans adjacent to a support have by far the greatest influence on the moment at that point. This leads to another method of finding moment

* Structural Engr., State Archt.'s Office, Madison, Wis.

† See diagrams in "Modern Framed Structures," Vol. II, "Statically Indeterminate Structures and Secondary Stresses," by Johnson, Bryan, and Turneaure, p. 35, or in "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, p. 299. These diagrams show influence lines for moments at interior supports of continuous beams.

in which only two spans are considered, these two spans having assumed values of end moments. The writer assumes the moment at the remote end of the first span to be $\frac{1}{12} w l^2$ if the second span from the column in question approximates the length of the first span. A value of $\frac{1}{24} w l^2$ is used for end spans. In general,

$$M_2 l_2 + 2 M_3 (l_2 + l_3) + M_4 l_3 = -\frac{1}{4} w_2 l_2^3 - \frac{1}{4} w_3 l_3^3$$

in which, M designates the moment at the support; l, the length of span; and w, the uniform load; each for the span as denoted by the subscripts.

If $M_2 = -\frac{1}{12} w_2 l_2^2$ and $M_4 = -\frac{1}{12} w_3 l_3^2$, then,

$$M_3 = -\frac{w_2 l_2^3 + w_3 l_3^3}{12 (l_2 + l_3)} \quad \ldots\ldots (8)$$

However, if $M_2 = -\frac{1}{24} w_2 l_2^2$ and $M_4 = -\frac{1}{24} w_3 l_3^2$, then,

$$M_3 = -\frac{5}{48} \frac{w_2 l_2^3 + w_3 l_3^3}{l_2 + l_3} \quad \ldots\ldots (9)$$

Let $w_2 = w_3$, $l = l_2$, the long span, and $n l = l_3$ the short span. If Equations (8) and (9) are to be expressed in inch-pounds instead of foot-pounds, then,

$$M_3 = -w l^2 \left(\frac{1 + n^3}{1 + n}\right) \quad \ldots\ldots (8a)$$

$$M_3 = -1.25 w l^2 \left(\frac{1 + n^3}{1 + n}\right) \quad \ldots\ldots (9a)$$

Now, let $C = \frac{1 + n^3}{1 + n}$. Then, again, in inch-pounds,

$$M_3 = -C w l^2 \quad \ldots\ldots (8b)$$

$$M_3 = -1.25 C w l^2 \quad \ldots\ldots (9b)$$

The relations between the long span, short span, and $C l^2$ are shown in Fig. 14. It will be noted that as far as this diagram is concerned the short span is never less than one-half the long span. The value of C is a maximum of 1.00 when $n = 0.00$ and 1.00; it is a minimum of 0.75 when $n = 0.5$; and it is symmetrical about the point at which $n = 0.5$. For instance, $C = 0.84$ when $n = 0.2$ or 0.8; and $C = 0.76$ when $n = 0.4$ or 0.6. Consequently, when the short span is less than one-half the long span, Fig. 14 may still be used by subtracting the short span from the long span to get an equivalent short span producing the same moment at the interior support. With a long span of 22 ft. and a short span of 8 ft., determine the value of $C l^2$ from Fig. 14 by subtracting 8 ft. from 22 ft. and using 14 ft. as the short span.

This method is approximate in that the moment at a column is influenced only by the adjacent spans and their end moments. A more accurate method is

to consider four spans, two on each side of the support at which the moment is desired.

Let l_1, l_2, l_3, and l_4 represent the spans carrying uniform loads per foot of w_1, w_2, w_3, and w_4 and let M_1, M_2, M_3, M_4, and M_5 be the moments at the five supports with M_3 as the moment the value of which is desired. If the continuous beam is restrained at each end of the series of four spans the value of M_3 will be greater than it would be with zero end moments. Therefore, the beam will be assumed as fixed at M_1 and M_5. The quantity, M_1, may be readily obtained by adding an auxiliary span, l_0, carrying a load, w_0, having an end moment, M_0, each having a value of zero.

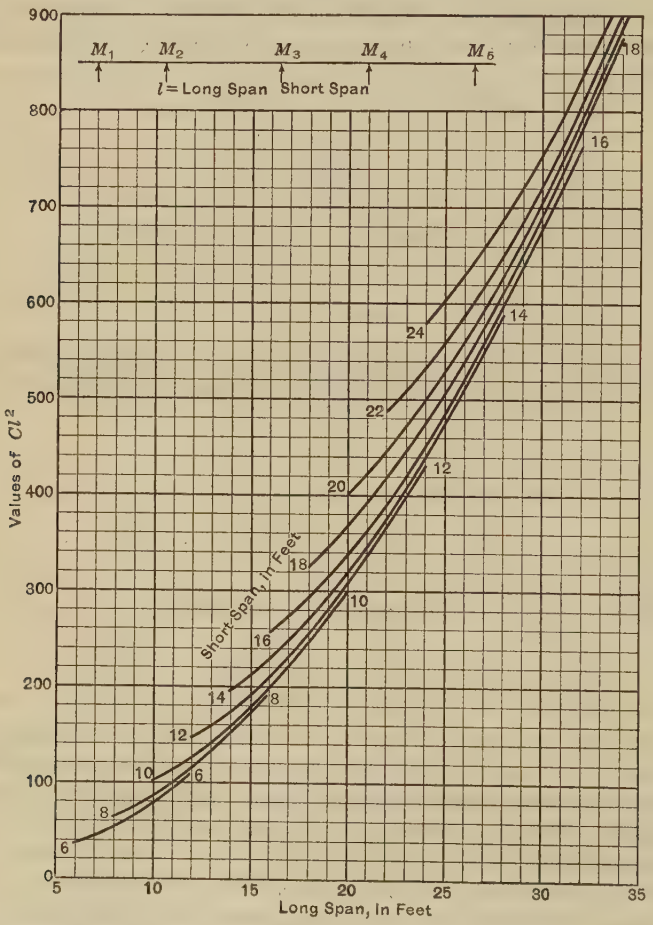

FIG. 14.

A perfectly general equation for the moment, M_3, can be obtained by applying the theorem of three moments to various pairs of spans and solving the resulting equations for M_3, which will be found to be,

$$M_3 = + \frac{a\, w_1\, l_1^3 - 2\,(1-a)\, w_2\, l_2^3 - 2\,(1-b)\, w_3\, l_3^3 + b\, w_4\, l_4^3}{8\,[l_2\,(2-a) + l_3\,(2-b)]} \quad \ldots\ldots(10)$$

In Equation (10), a and b are constants for a given case, as follows:

$$a = \frac{l_2}{1.5\ l_1 + 2\ l_2} \text{ and } b = \frac{l_3}{2\ l_3 + 1.5\ l_4}$$

For all ordinary cases Equation (10) gives a negative result because the negative terms of the numerator are usually much larger than the positive terms.

Illustrative Problem.—Find the moment at the center support of a continuous beam, in which $l_1 = 8$ ft., $l_2 = 20$ ft., $l_3 = 12$ ft., $l_4 = 15$ ft., and $w = 2\,500$ lb. per lin. ft. for all spans. If Fig. 14 is used, l_2 and l_3 only are considered giving a value of 305 for $C\,l^2$ and a moment of $305\ (2\,500) = -762\,500$ in-lb. for M_3.

If Equation (10) is used,

$$a = \frac{20}{1.5\ (8) + 2\ (20)} = 0.385 \text{ and } b = \frac{12}{2\ (12) + 1.5\ (15)} = 0.258$$

Then,

$$M_3 = 2\,500 \left(\frac{0.385\ (8)^3 - 2\ (1 - 0.385)\ (20)^3 - 2\ (1 - 0.258)(12)^3 + 0.258\ (15)^3}{8\ [20\ (2 - 0.385) + 12\ (2 - 0.258)]} \right)$$

$$= -66\,500 \text{ ft-lb.} = -798\,000 \text{ in-lb.}$$

Now, change l_1 to 15 ft. and l_4 to 8 ft., but leave the inner spans as before and solve for M_3 by Equation (10). Then,

$$a = \frac{20}{1.5\ (15) + 2\ (20)} = 0.320 \text{ and } b = \frac{12}{2\ (12) + 1.5\ (8)} = 0.333$$

$$M_3 = 2\,500 \left(\frac{0.320\ (15)^3 - 2\ (1 - 0.320)\ (20)^3 - 2\ (1 - 0.333)\ (12)^3 + 0.333\ (8)^3}{8\ [20\ (2 - 0.320) + 12\ (2 - 0.333)]} \right)$$

$$= -69\,500 \text{ ft-lb.} = -834\,000 \text{ in-lb.}$$

In the first case the answer obtained by the use of Fig. 14 is 4.5 per cent. less than that obtained by Equation (10), while in the second case the difference is 9.5 per cent. The first error is relatively small, but the second is large enough to require correction. It should be noted that in the latter problem l_1 is less than l_2 and also that l_4 is less than l_3. Hence, the moments, M_2 and M_4, are less than those assumed in Fig. 14 and, consequently, M_3 is greater than that given by this diagram. Therefore, it may be concluded that when l_2 exceeds l_1 and l_3 exceeds l_4 by a considerable proportion the value of M_3 obtained by using Fig. 14 should be increased by about 10 per cent. Likewise, when l_1 exceeds l_2 and l_4 exceeds l_3 by a considerable proportion the value of M_3 given by Fig. 14 may be reduced by about 10 per cent.

It should be noted that in Equation (10), different values of w can be used in the different spans, thus allowing the placement of live load and dead load on the two inner spans with dead load only on the two outer spans. In this way the maximum value of M_3 is produced.

Stirrups.—In the design of shear reinforcement for reinforced concrete beams the writer has devised a few diagrams for his own use which may be of interest to other engineers. Here, again, he has tried to get practical results with a maximum of speed and a high degree of accuracy. The diagrams illustrated herewith (Fig. 15, (*a*), (*b*), and (*c*)) are for three different beam

widths 8, 12, and 16 in. and for ⅜-in., round, single-loop stirrups at a unit stress of 16 000 lb. per sq. in., but the general principles could be used in the construction of diagrams to fit other conditions.

It is the writer's practice to choose the values of b and d for a series of beams to satisfy the maximum conditions of moment and shear and then keep these dimensions constant for the group.

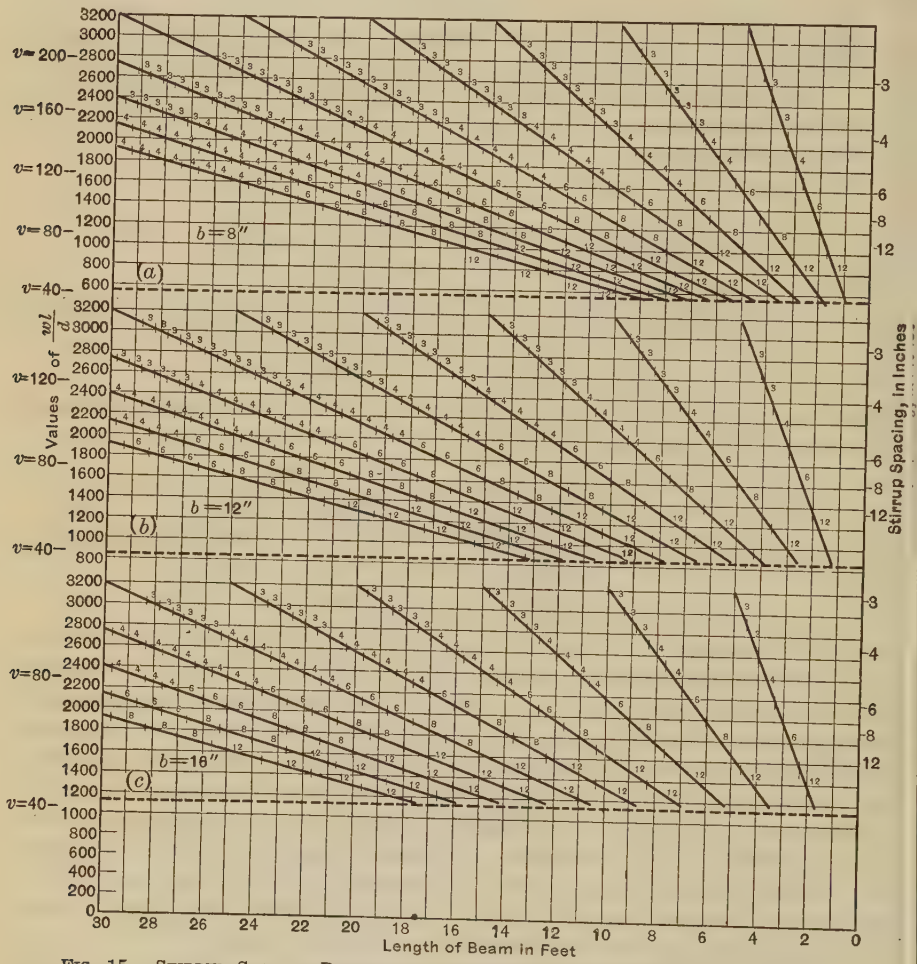

FIG. 15.—STIRRUP SPACING DIAGRAMS FOR ⅜-INCH ROUND, SINGLE-LOOP STIRRUPS, f_s EQUALS 16 000 POUNDS PER SQUARE INCH.

For a beam carrying a uniform load per foot, the end shear, V, equals $\frac{w\,l}{2}$, part of which, $v_2\,b\,j\,d$, is taken by the concrete, and the remainder, V', by the stirrups. The stirrup spacing, s, equals $\frac{A_s f_s j\,d}{V'}$, or,

$$\frac{V'}{d} = \frac{A_s f_s j}{s} = \frac{2\ (0.1104)\ 16\,000\ (0.875)}{s} = \frac{3\,090}{s}$$

for $\frac{3}{8}$-in. round stirrups. Then,

$$\frac{V}{d} = \frac{V'}{d} + \frac{v_2\, b\, j\, d}{d}$$

or, with $v_2 = 40$ lb. per sq. in., and $j = 0.875$,

$$\frac{V}{d} = \frac{w\, l}{2\, d} = \frac{3\,090}{s} + 35\, b$$

or,

$$\frac{w\, l}{d} = \frac{6\,180}{s} + 70\, b$$

In the construction of each diagram a dotted line was drawn parallel to the base as shown in Fig. 15 (*e*) and at a distance of 70 *b* above it. Other parallel lines were drawn above this at a distance equal to $\frac{6\,180}{s}$ for different stirrup spacings.

In some of his earlier diagrams the writer stopped here without indicating any diagonal lines with selected stirrup spacings. A horizontal scale giving the half-span in inches was used and any particular problem was solved by finding a point in the diagram at the intersection of this length with the value of $\frac{w\, l}{d}$. This point was connected with the span center at the lower right-hand corner (see Fig. 15 (*c*)) by means of a straight edge and the required stirrup spacing was read along the straight edge.

Recently, it has been found advantageous to indicate some diagonal lines (Fig. 15) with well-chosen stirrup spacings and to use as a horizontal scale the span of the beam in feet. Obviously, no stirrups are needed beyond the point where the diagonal lines intersect the dotted horizontal line. In the region where stirrups are needed the spacing should not be allowed to exceed one-half to three-fourths the effective depth of the beam.

Illustrative Problem.—A reinforced concrete beam, 18 ft. long, having a width of 12 in. and an effective depth of 16 in., carries a load of 2 000 lb. per lin. ft. Find the spacing of $\frac{3}{8}$-in., round, single-loop stirrups having a unit stress of 16 000 lb. per sq. in.

With a value of $\frac{w\, l}{d} = \frac{2\,000\ (18)}{16} = 2\,250$, and a beam length of 18 ft., Fig. 15 (*c*) gives the following stirrup spacing, in inches: 4, 4, 4, 4, 6, 6, 8, 8, 12, 12.

F. Theodore Mavis,* Assoc. M. Am. Soc. C. E. (by letter).—In his opening paragraph the author states that,

"Although reinforced concrete structures have been used extensively for the past twenty-five years, and the general theory of their design has been quite fully developed, yet engineering literature, including the publications of the Society, give meager discussions of the proper distribution of reinforcing steel in concrete beams and slabs."

* Urbana, Ill.

This charge seems to be a bit broad when one considers the valuable papers and discussions dealing with the analysis of statically indeterminate structures published by the Society from time to time. Any one of the many variations in the application of the elastic theory arrives at the moments, thrusts, and shears acting at any section by a rational method.

The author considers the case of a series of identical spans, each loaded with the same uniform load, w lb. per ft., and arrives at a set of rules governing the relative amounts of steel required at the center of a given span and at the supports. This is an elementary problem in mechanics, and one which should not give the structural engineer the slightest cause for worry. Almost any textbook in mechanics has a table or chart of moment coefficients at the supports for a structure of this kind having from two to six or more spans.

All this is of no value to the structural engineer who is dealing (1) with girders carrying beam loads; (2) with frames made up of beams or girders which are not of equal length; (3) with structures carrying a uniform load in one span differing from that in another; (4) with structures carrying machinery or moving concentrated loads; or (5) with structures having haunched beams or girders. Handbooks and textbooks on reinforced concrete contain much more information of value in the design of even ordinary structures than that presented in the paper.

The author states that the total reinforcement required by the formula, $M = \frac{1}{8} w l^2$, may be divided in any manner between positive steel at the center and negative steel at the supports. The writer assumes that a monolithic concrete structure built in accordance with a satisfactory design is one that will not develop serious cracks when it is subjected to the loads for which it was designed. Consequently, he can hardly agree that placing all the steel in the top or all the steel in the bottom of a beam continuous over a support would make even a "theoretically satisfactory design".

In the design of a continuous framed structure the engineer is faced with a problem similar to the one which is met in every structural design: (1) He must know the dimensions of every part of the structure before he can analyze it; and (2) he must analyze the structure before he can design the sections.

Assume that the designer has selected the sizes of members in the structure, based on preliminary calculations and on his experience and judgment. This first draft of the design should, for economy of time and labor, approximate the final design. Now, assuming that the structure "hangs together", he can make an analysis of it under critical conditions of loading, and determine the moments, thrusts, and shears acting at any section. As long as the structure behaves as an unbroken elastic body, these moments, thrusts, and shears obtain, regardless of the amount or location of steel reinforcement. Knowing the internal forces at any section, the engineer can design that section to resist these forces, placing the required amounts of steel in the various members to "take" tension and prevent cracks from opening. Evidently, the author does not question the conventional methods for calculating the amount

of steel required in a given section when the bending moments and shears are known.

Methods of analyzing a statically indeterminate frame are well known and an elastic analysis of an important structure takes a relatively short time. Even a rough analysis would give a basis of design which is much more reliable and more widely applicable than the restricted set of rules laid down by the author.

The author further states that the bending produced in a column is a question of the amount of negative reinforcement in the beams which frame into that column. In what way does the bending moment in the columns depend on the amount of steel in the top of the beams, provided cracks do not open in the structure? As long as cracks do not occur, the bending moment at a given section depends only on the loads and the elastic properties of the structure, and will be the same for a given loading regardless of the amount of reinforcement in the top of the beam at a support. In this case also, would it not be better to analyze the structure as an unbroken elastic body and design the members to resist the internal stresses determined in such an analysis, placing steel where necessary to "take" tension?

Rules such as those laid down by the author are dangerous in the extreme. Unless an engineer is qualified to analyze a structure to carry the loads coming on it he had better not quibble over the difference between the steel required by such formulas as $M = \frac{1}{8}\ w\ l^2$ and $M = \frac{1}{6}\ w\ l^2$. In such a case it is better that he "play safe", or else leave indeterminate structures, which include monolithic concrete structures, alone.

Jacob Feld,* Assoc. M. Am. Soc. C. E. (by letter).—Doubtless, in the great majority of concrete designs not enough attention is paid to the proper distribution as well as amount of reinforcing steel. The complete analysis of a reinforced concrete structure, even so simple a member as a beam, is so tedious and complicated that the use of several empirical simplifications has become practically universal. So common has this become that only a few of the many who use standard methods of reinforced concrete design realize that there are approximations inherent in the formulas; and far too many know little of the principles involved.

Careful analysis of a reinforced concrete design almost invariably points to possibilities of large savings in reinforcement. At the same time, the amount of work and time required for such analysis and the additional field cost to follow special and unusual details make it (the writer believes) in the majority of designs uneconomical for the owner and contractor to deviate from present practice.

The writer recalls one instance which will well illustrate this belief. In detailing the steel for a flat slab floor, it is customary to continue all panel and column band rods (in the top and the bottom of the slab) past the line of inflection a distance at least equal to that required to develop the full ten-

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sion value of the rod by bond. This is equivalent to the requirement that the full value of all rods be required on each side of the line of inflection; in other words, that the full positive and negative bending moments be taken care of at opposite sides of each line of inflection. This is evidently too conservative an assumption. By cutting each rod 12 in. shorter and shifting alternate rods right and left of the line of inflection by 6 in., a saving of more than 5% in weight of reinforcing steel is effected; and at each line of inflection one-half the maximum moments can be safely taken care of—certainly sufficient for a point of zero theoretical moment. To allow for a variation in location of the lines of inflection, due to non-symmetrical loading or other causes, the bends in the rods are staggered alternately 6 in. each side of the lines of inflection. In this manner, all rods are identical as to lengths and locations of bends—an ideal condition for the project for which all rods are ordered bent to shape or bent on the bench.

However, as is often the case, the band rods are first placed in position on the forms and bent into shape *in situ*. Considerable extra time and work are then required to place the rods in the proper staggered position and to stagger the bends. It is doubtful whether the saving in steel compensates for the extra labor. The writer, therefore, only uses this refinement in detail where the size of rods is so heavy that the bending must be done at the bench or when he is certain that bent rods will be ordered for the job.

Mr. Myers' analysis of maximum bending moments in the section entitled "General Principles" is in error because he has assumed that the elastic curve of a continuous or restrained beam can be cut into sections which are identical with the elastic curves of simple and cantilever spans. That this is not the case can be seen from the following analysis.

In an elastic structure, such as a beam, the curve of deformation must be continuous and of the second degree. There can be no cusp points, that is, at each point there can be only one slope to the curve. In Mr. Myers' solution, the slope of the "simple span" at the point of inflection is not the same as the slope of the "cantilever" at the same point.

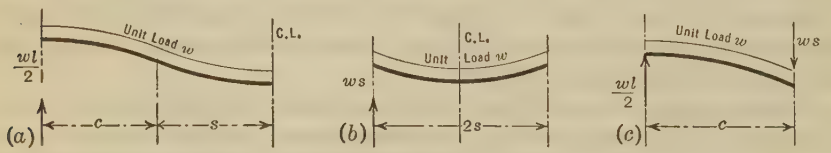

FIG. 16.

The simple span, in the general case shown in the Fig. 16 (*b*), of Span 2*s*, end reaction of $w\ s$, has a slope at the end of $-\frac{w\ s^3}{3\ E\ I}$. In order that the curve of the continuous beam be smooth, this slope must be equal and opposite in sign to the slope at the end of the cantilever, shown in Fig. 16 (*c*). The latter slope is $\frac{w\ c^3}{3\ E\ I} + \frac{s\ w\ c^2}{2\ E\ I}$.

Equating and solving:

$$\frac{s^3}{c^3} - \frac{3\,s}{2\,c} - l = 0$$

or,

$$s = \frac{3\,c}{2} \text{ (approx.)}; \; c = 0.2\,l; \text{ and } s = 0.3\,l.$$

The approximate solution is about 2% in error. Only for the case of $c = 0.2\,l$, that is, when the point of inflection is at the 0.2 point of the span, is Mr. Myers' assumption correct.

To check this solution, the deflections of the beam can be investigated. A continuous beam has a maximum deflection of $\frac{w\,l^4}{384\,E\,I}$. The deflection of the center simple span $(0.6\,l)$ is:

$$\frac{5\,(0.6)^4\,w\,l^4}{384\,E\,I}$$

and of the cantilever of Span $0.2\,l$, loaded uniformly, is:

$$\frac{(0.2)^4\,w\,l^4}{8\,E\,I}$$

and of the cantilever loaded with the reaction of the simple span as a concentration at the free end, is:

$$\frac{(0.3)\,(0.2)^3\,w\,l^4}{3\,E\,I}$$

Summing the three deflections (this is permissible if all the strains are within the elastic limit), the total deflection is:

$$\frac{215\,w\,l^4}{80\,000\,E\,I}, \text{ or } \frac{w\,l^4}{377\,E\,I}$$

so that the approximate location of the point of inflection at the 0.2 span point gives a deflection only $\frac{7}{384}$, or less than 2% in error. This is the basis of the empirical rule for the location of bends in reinforcement of continuous beams and slabs, including flat slabs, at the 0.2 point of the spans.

The entire analysis of Mr. Myers falls down on the removal of the false assumption of beam action.

The method described in the paper for the determination of the number and spacing of stirrups is so long and tedious that it can hardly become of common use. The writer has developed some short cuts in concrete design for his own purposes and uses the following method for designing stirrups, which he believes is much simpler. As far as he knows, the method is original, although he claims no new ideas.

The standard formula (Equation (3)), is used as the basis of the method:

$$s = \frac{A_v f_s j d}{V_1}$$

But,

$$V_1 = (v - v_2)\ b\ j\ d$$

in which, v is the unit shear.
Hence,

$$s = \frac{A_v\ f_s\ j\ d}{(v - v_2)\ b\ j\ d} = \frac{A_v\ f_s}{(v - v_2)\ b}$$

a simple and easily remembered as well as extremely useful form. There are a limited number of stirrup rod sizes; usually only $\frac{3}{8}$ and $\frac{1}{2}$-in. round rods are used. According to any one building code, the value of f_s is constant, and the concrete is assumed to take care of a certain unit shear.

For a $\frac{3}{8}$-in. rod, A_v is 2×0.110, f_s is 16 000, and v_2 is 40. By substituting, there results:

$$s = \frac{440}{v - 40}$$

From a shear diagram or from mental calculations, the v at any point in the beam can be found; then by slipping the slide of the slide-rule, with the indicator at 440, the various successive stirrup spacings are read directly. It is well to remember that a formula giving the total number of stirrups necessary cannot take into account the requirement that the maximum stirrup spacing is not to exceed some fraction of the beam depth, say, one-half or three-quarters (the usual values specified). A simple straight-line diagram can be used to determine the part of each span, for any definite end shear, which must be reinforced with stirrups.

Charles S. Whitney,* M. Am. Soc. C. E. (by letter).—The principles given in this paper appear to be based on erroneous premises. They are contrary to generally accepted practice and the burden of the proof, therefore, should be on the author. Both experimental and theoretical investigations have established that the theory of elastic frames is applicable to reinforced concrete construction, and that it is the most practical method of design. For the results of the application of this theory the author would substitute arbitrary rules which give very different results.

In developing his rules, Mr. Myers has made two fundamental errors: First, the variation of the moments of inertia in T-beams does not prevent the satisfactory application of the theory of elasticity; and, second, in establishing bending moment coefficients, he has considered only the case of full load on all spans. With partial loading, the sum of the maximum moments at the center and supports may exceed the simple span moment.

Ten or fifteen years ago some designers were using such rules as those suggested by the author with apparent success; but that does not indicate that structures so designed would prove as efficient under full load as those designed in accordance with the latest rulings of the Joint Committee on Specifications for Concrete and Reinforced Concrete.

David A. Molitor,† M. Am. Soc. C. E. (by letter).—The author has presented a subject that deserves careful consideration because it deals with

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† Cons. Engr., Detroit, Mich.

several of the mooted questions of designing reinforced concrete beams with restrained ends as these questions arise in the realm of building construction.

For concrete beams of rectangular cross-section, simply supported, but continuous over their supports, and subject to variable moving loads, the Theorem of Three Moments, which is based on the Theory of Elasticity, will undoubtedly lead to the most economic designs. Such beams may be used in smaller bridges, but rarely find application in monolithic building construction where most beams have a T-shape.

The T-shaped beam in buildings is strictly monolithic with columns and floor-slabs, and reinforcing steel so interlocked as to prevent unsightly cracks. The superimposed loads are of extremely variable character, uniformly distributed dead and live loads, and various concentrated loads, all of which may attain the maximum values assumed in the design, or they may be greatly reduced when there is no live load. It would be futile to attempt a rigid analysis of such a beam, or system of beams, in accordance with the theory of elasticity, and hence the problem is usually solved by some approximate method.

The earlier attempts in this direction were naturally patterned after the idea of taking account of restraint, first, of the beams themselves where they are continuous over supports, and, second, by fixity at column connections. This was prompted by the appearance of cracks whenever inadequate steel reinforcement was provided at points of restraint.

Since the live load may be applied to any spans, or portions of spans, of a continuous system of beams for the purpose of design, the following approximate moments were used, $\frac{w\,l^2}{10}$ for the positive center and negative end moments of an end span (restrained at one end only), and $\frac{w\,l^2}{12}$ for the positive center and negative end moments of all interior spans (restrained at both ends). These results follow from an application of the Theorem of Three Moments for maximum load effects, assuming a uniform moment of inertia throughout. The maximum sum of the end and center moments of an interior span is thus $\frac{w\,l^2}{6}$, while the theoretical minimum for this sum is only $\frac{w\,l^2}{8}$ for a case of simultaneous loading over all spans. For end spans this sum is $\frac{w\,l^2}{5}$, while in reality it is not a constant, but must vary, depending on the point of contra-flexure. According to this method of design the end moments at a column, between an end span and an adjacent interior span, are usually unequal, so that such a column itself would be subjected to bending.

While this rule is simple when applied to rectangular beams, for which it was actually intended, difficulties are encountered in its application to sections of T-form. The T-beam is uneconomical when designed for equal end and center moments because, for a correctly designed section at the center, where the flange (slab) is adequate for the compression area and the

stem and steel are just sufficient for the tensile stress, the end section of the beam will be incapable of taking the compression on the lower side which now falls to the stem area, and compressive steel must be provided or the stem must be haunched. This difficulty can be avoided by reducing the end moments and increasing the center moments. Thus, the portions subjected to positive moment will be lengthened to develop the available concrete on top, and the portions subjected to negative moment will be shortened as limited by the available concrete in the stem only.

Now, if these moments are combined in some arbitrary manner, at the same time retaining a sum moment of $\frac{w\,l^2}{6}$, for an interior span, the laws of continuity as established by the elastic properties of the materials will be violated unless due account is taken of this violation in the design of the beam. This may best be accomplished by ignoring the theoretical moment distribution indicated for continuity and treating the chain of spans as a series of cantilever systems in which the several points of contraflexure may be regarded as arbitrarily assigned hinged points; but unless hinges are actually provided at these points of contraflexure, such cantilever systems may not act in accordance with the assumptions, and the position of each hinge may change to accommodate itself to the various positions of the live load on the several spans.

Take, for example, any interior span having a certain definite loading and restrained by known end moments from adjacent spans. Then, the points of contraflexure are at once fixed and the corresponding center moment is determined, making the sum of this center moment and the mean of the two end moments (if unequal), equal to the theoretical minimum of $\frac{w\,l^2}{8}$. Now, reduce the end moments by unloading the side spans, and the points of contraflexure will shift accordingly to new positions, resulting in a larger center moment but again making the sum of the center moment and the mean of the two new end moments equal to $\frac{w\,l^2}{8}$. Thus, within limits of possibility in shifting live loads, the center moment will attain a maximum value when the corresponding end moments are minima; and, conversely, the center moment will become minimum when the two end moments are maxima. Now the sum of the maximum values of the end and center moments is necessarily greater than $\frac{w\,l^2}{8}$.

Since a beam must be capable of sustaining the maximum load effects at all points with equal safety, therefore, the author is seriously in error when he bases his design on the minimum sum of $\frac{w\,l^2}{8}$, which could be correct only for a single simultaneous condition of uniform loading over all spans.

The question then resolves itself into assigning a reasonable, but safe, value to the sum of center and mean end moments of an interior span, such

that the shift in points of contraflexure may not exceed permissible limits. Obviously, the value, $\frac{w\,l^2}{8} = 0.125\,w\,l^2$, is too small and the value, $\frac{w\,l^2}{6} = 0.167\,w\,l^2$, is rather large, because it is based on a ratio of live load to dead load that is quite out of proportion for buildings and was originally chosen to fit bridge loadings with relatively high live loads.

The 1919 Building Code of the City of Detroit, Mich., specifies that, for beams supported partly or entirely by cantilever action, the sum of the negative and positive bending moments, taken together, shall not be less than $\frac{6}{5} = 1.2$ of the bending moment for non-continuous members, and the assumed points of inflection of such special cantilever construction shall be clearly marked. This practice has been followed by progressive engineers in Detroit during the past ten years.

Since the code does not distinguish between end and interior spans, it must be presumed that the 20% increase applies to both. Also, since this increase is applied to the sum of the end negative and center positive moments, it must apply, therefore, to each of the two moments separately. Accordingly, for interior spans, the sum of the negative and positive moments becomes $1.2 \times 0.125\ w\,l^2 = 0.15\ w\,l^2$. For end spans this sum is not a constant but increases as the point of contraflexure is placed farther out from the restrained end. The 20% increase for end spans is, therefore, even more necessary than for interior spans.

The subject is here presented in simple form for practical use according to the writer's views, giving formulas (see Fig. 17 and Table 1) and a table of moments (Table 2). The formulas on the left-hand side of Table 1 were derived for cantilever spans having all spans simultaneously loaded with a uniform load, w, per foot of span. The 20% increase was then applied (right-hand side of Table 1) to allow for variations in the points of contraflexure as affected by different cases of loading, and the formulas thus obtained were used to compute Table 2 giving end and center moments.

It should be noted that the locus of the point of contraflexure for an interior span is the parabola representing the constant sum moment, $0.15\,w\,l^2$, while for an end span the locus of the point, C, is a straight line (Fig. 17) such that for every point, C, there will be a separate parobola passing through it to represent the end and center moments.

Fig. 17 is drawn for one particular set of moments, namely, $M_c = 0.08\,w\,l^2$ and $M_e = 0.07\,w\,l^2$, with $c = 0.135\,l$ for the interior span; and $M_c = 0.117\,w\,l^2$ and $M_e = 0.07\,w\,l^2$, with $c = 0.117\,l$ for the end span. This makes M_e equal for both spans, and no moment is transmitted to the column between the two spans. The same feature may be preserved even when the spans are of unequal lengths or when the loads are widely different on the two spans, as will be illustrated by a problem.

Table 2 is self-explanatory and will suffice for all spans of variable lengths and all cases of loading. The numerical moment coefficients, $\frac{1}{m}$, are to be

multiplied by $w\,l^2$ to give actual moments, and the values of $\frac{c}{l}$ for points of contraflexure are to be multiplied by l to give the actual distances, c.

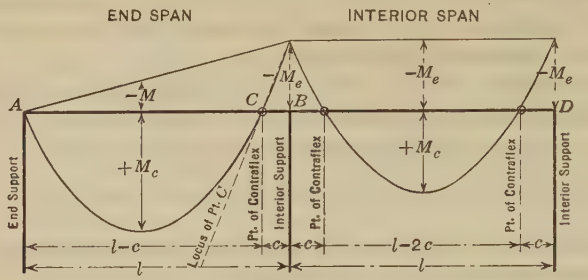

FIG. 17.

TABLE 1.—FORMULAS FOR CANTILEVER BEAMS, UNIFORMLY LOADED (SEE FIG. 17).

MOMENTS FOR ALL SPANS LOADED. INTERIOR SPANS.	MOMENTS INCREASED 20%. INTERIOR SPANS.
$M_c = \frac{w}{8}\left(l - 2\,c\right)^2$	$M_c = 0.15\ w\ (l - 2\,c)^2$
$M_e = \frac{w\,c}{2}\left(l - 2\,c\right) + \frac{w\,c^2}{2} = \frac{w\,c}{2}\left(l - c\right)$	$M_e = 0.6\ w\ c\ (l - c)$
$M_c + M_e = \frac{w\,l^2}{8} = \text{constant}$	$M_c + M_e = 0.15\ w\ l^2 = \text{constant}$
$c = \frac{l}{2} - \sqrt{\frac{l^2}{4} - \frac{2\,M_e}{w}} = \frac{l}{2} - \frac{1}{2}\sqrt{\frac{8\,M_c}{w}}$	$c = \frac{l}{2} - \sqrt{\frac{l^2}{4} - \frac{M_e}{0.6\,w}} = \frac{l}{2} - \frac{1}{2}\sqrt{\frac{M_c}{0.15\,w}}$
END SPANS.	**END SPANS.**
$M_c = \frac{w}{8}\left(l - c\right)^2 = \left(1 - \frac{c}{l}\right)^2 \frac{w\,l^2}{8}$	$M_c = 0.15\left(1 - \frac{c}{l}\right)^2 w\,l^2$
$M_e = \frac{w\,c}{2}\left(l - c\right) + \frac{w\,c^2}{2} = \frac{w\,c\,l}{2}$	$M_e = 0.6\ w\,c\,l$
$M = \frac{M_e}{2\,l}\left(l - c\right) = \frac{w\,c}{4}\left(l - c\right)$	$M = \frac{0.6\ M_e}{l}\ (l - c) = 0.3\ w\ c\ (l - c)$
$M + M_c = \frac{w}{8}\left(l^2 - c^2\right)$	$M + M_c = 0.15\ w\ (l^2 - c^2)$
$M_c + M_e = \frac{w}{8}\left(l + c\right)^2 = \text{variable}$	$M_c + M_e = 0.15\ w\ (l + c)^2 = \text{variable}$
$c = \frac{M_e}{0.5\,w\,l}$	$c = \frac{M_e}{0.6\ w\,l}$

Now it may be contended that, because the points of contraflexure are treated as imaginary hinges of a cantilever system, which hinges have not actually been provided in the construction, it is not certain that the structure will behave in accordance with the assumptions. This point can be answered as follows: In the first place, any of the various assumptions that may be made as to moment distribution in this analysis represents possible conditions for some cantilever system, and if the material (both steel and concrete) at any section of a beam is ample to care for the moment assigned to it, each

such section must at least do its share of the total work to be performed. Furthermore, the possible shift of a point of contraflexure can be reduced to a minimum limit by bending up part of the positive steel so as to have the first bent bar intersect the beam axis in the point of contraflexure. Some negative steel must also extend out from the support to the adjacent points of contraflexure and for a distance of at least $\frac{d}{2}$ beyond these points.

TABLE 2.—End and Center Moments for Continuous Beams.*

End Spans.					Interior Spans.				
End Moment.		Center Moment.		Values of $\frac{c}{l}$.	End Moment.		Center Moment.		Values of $\frac{c}{l}$.
Values of m.	Values of $\frac{1}{m}$.	Values of m.	Values of $\frac{1}{m}$.		Values of m.	Values of $\frac{1}{m}$.	Values of m.	Values of $\frac{1}{m}$.	
19.2	0.0522	8	0.1250	0.087	40	0.0250	8	0.1250	0.045
18.5	0.0540	8.05	0.1242	0.090	35.1	0.0285	8.25	0.1215	0.050
17.5	0.0570	8.15	0.1229	0.095	32.0	0.0312	8.40	0.1188	0.055
16.7	0.0600	8.25	0.1215	0.100	29.4	0.0340	8.63	0.1160	0.060
15.9	0.0630	8.33	0.1202	0.105	27.4	0.0365	8.82	0.1135	0.065
15.1	0.0660	8.43	0.1188	0.110	25.6	0.0391	9.03	0.1109	0.070
14.5	0.0690	8.52	0.1175	0.115	24.1	0.0416	9.23	0.1084	0.075
13.9	0.0720	8.62	0.1162	0.120	22.7	0.0440	9.45	0.1060	0.080
13.3	0.0750	8.73	0.1148	0.125	21.5	0.0467	9.68	0.1033	0.085
12.8	0.0780	8.82	0.1135	0.130	20.3	0.0492	9.93	0.1008	0.090
11.9	0.0840	9.04	0.1109	0.140	19.4	0.0516	10.2	0.0984	0.095
11.1	0.0900	9.24	0.1084	0.150	18.5	0.0540	10.4	0.0960	0.100
10.4	0.0960	9.38	0.1058	0.160	17.7	0.0564	10.7	0.0936	0.105
9.80	0.1020	9.69	0.1033	0.170	17.0	0.0587	10.9	0.0913	0.110
9.25	0.1080	9.92	0.1009	0.180	16.4	0.0610	11.2	0.0890	0.115
8.78	0.1140	10.15	0.0985	0.190	15.7	0.0634	11.5	0.0866	0.120
8.33	0.1200	10.4	0.0960	0.200	15.3	0.0655	11.8	0.0845	0.125
6.67	0.1500	11.8	0.0845	0.25	14.7	0.0678	12.1	0.0822	0.130
5.55	0.180	13.6	0.0735	0.30	14.3	0.0700	12.5	0.0800	0.135
4.76	0.210	15.7	0.0635	0.35	13.8	0.0723	12.9	0.0777	0.140
4.16	0.240	18.5	0.0540	0.40	13.4	0.0744	13.2	0.0756	0.145
3.70	0.270	22.0	0.0454	0.45	13.1	0.0765	13.6	0.0735	0.150
3.33	0.300	25.7	0.0375	0.50	12.7	0.0786	14.0	0.0714	0.155
2.78	0.360	41.6	0.0240	0.60	12.4	0.0805	14.4	0.0695	0.160
2.38	0.420	74	0.0135	0.70	12.1	0.0828	14.9	0.0672	0.165
2.08	0.480	167	0.0060	0.80	11.8	0.0846	15.3	0.0654	0.170
1.85	0.540	666	0.0015	0.90	11.6	0.0866	15.8	0.0634	0.175
1.67	0.600	∞	0.0000	1.00	11.4	0.0875	16.0	0.0625	0.177

*The center moment need never exceed $\frac{w\,l^2}{8}$ for any span.

For interior spans the center moment shall be not less than $\frac{w\,l^2}{16}$, and the end moment shall be not less than $\frac{w\,l^2}{40}$.

For end spans the end moment shall be not less than $\frac{w\,l^2}{20}$, and may be anything, depending on adjacent spans.

The quantity, m, is the coefficient in the moment equation, such that $M = \frac{1}{m}\,w\,l^2$.

Also, since the loads applied to a beam will first stress the central portion containing the positive steel before the load effect can reach the supports, it follows that the positive steel must work to capacity, delivering to the cantilever ends only that portion of the load in excess of its capacity and for which the negative steel makes ample provision.

On the other hand, enough negative steel may be provided to carry the entire load on cantilever arms, thus entirely unloading the central portion where little or no positive steel would then be required. However, this alternative is not utilized except in special cases of short interior spans or short end spans when the end moments from the longer adjacent spans are proportionately large.

Tests prove that continuous beams, if designed according to the theory of elasticity, will develop moments and stresses closely approaching the designed values, especially within the range of strict proportionality, until the elastic limit is reached, and even beyond this range possibly to the point of rupture.

That does not prove (as many assume it does) that for beams designed otherwise the results under tests would disagree materially with the assumptions on which the design was based. In fact, the few tests reported by Professor Moersch distinctly support this latter view. However, the behavior within the range of proportionality now becomes somewhat different for a structure, but the ultimate behavior at rupture again agrees with the variant assumptions, the same as for beams designed by the theory of elasticity. In practice, this is verified by innumerable observations, although tests made to substantiate this fact are quite limited. Future experiments along these lines would be very valuable.

After the elastic limit is exceeded, the elastic theory no longer governs, and entirely new relations are set up. For this there is abundant evidence which has not been given proper consideration by theorists, although the facts are quite well known to practice.

Advantage can be taken of the ductility of steel only when dealing with structures involving redundancy, and since continuous beams belong to this class, it is proper to give consideration to this steel characteristic.

Take, for instance, a redundant tension member in a steel frame or truss, and suppose that its length is shorter than intended, and the member is forced into place and its ends are then permanently connected. When the frame receives its maximum load, this particular member will be overstressed long before the other members attain their calculated stresses. The stress in such a redundant member may readily exceed the elastic limit, and, if this happens, this member simply stretches a small amount (permanently) and thus adjusts itself to the defects in construction without producing any permanent damage. Thereafter, for repeated applications of the maximum load, the member will function normally without ever again being stressed beyond its yield point.

This is a well known phenomenon, but, strangely enough, it has received little consideration in practice. A better understanding of this subject would relieve most doubts now entertained by many engineers with regard to statically indeterminate structures.

Ordinary steel elongates about 25% of its length at the ultimate stress, while the elongation at the elastic limit amounts to less than 0.1%, or $\frac{1}{250}$ part of the total elongation. If then some of the steel which is stressed beyond the elastic limit is permanently stretched 1 or 2%, due to overloads which have stressed steel in other parts of the structure (where there is redundancy)

merely to the elastic limit, then the ultimate condition would be all steel stressed equally after re-adjustment of stress has once been accomplished. In the case of a reinforced concrete beam all this can take place without showing visible cracks over the supports. When cracks do appear it is due to much greater overloads, approaching the ultimate, or to other causes attributable to methods of construction.

The following example is given to illustrate the method of designing cantilever beams in accordance with this outline. Three tables (not here reproduced) are useful in designing concrete beams. They are (1) table of moments of resistance ($M\,R$) of rectangular concrete beams for various widths and depths according to the formula, $M\,R = \frac{R_c\,b\,d^2}{12}$ ft-lb., in which, the width, b, and the depth, d, are in inches, and $R_c = \frac{1}{2}\,f_c\,k\,j$; (2) table of moment of resistance per 1 in. width of T for various effective depths of beam; and (3) moment of resistance in foot-pounds for 1 sq. in. of compression steel for various effective depths of a beam.

Example.—Three spans have uniform loads of 2 200 lb. per lin. ft., and concentrated loads as shown in Fig. 18. Assume that $f_c = 750$ lb. per sq. in.; $f_s = 20\,000$ lb. per sq. in. for longitudinal steel and 16 000 lb. per sq. in. for stirrups; $n = 15$; $R_c = \frac{1}{2}\,f_c\,k\,j = 118.8$; and $M\,R = \frac{118.8}{12}\,b\,d^2$ ft-lb. for concrete sections. The shear on the reinforced section may be as much as 120 lb. per sq. in. The beam has a 4-in. solid slab flush with the top and will act as a T-beam.

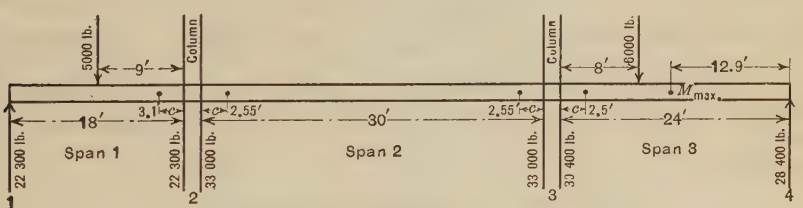

FIG. 18.

Maximum moment, Span 1:

$$2\,200 \times \frac{18^2}{8} + 5\,000 \times \frac{18}{4} = 111\,800 \text{ ft-lb.}; \quad w\,l^2 = 8 \times 111\,800 = 894\,000 \text{ ft-lb.}$$

Maximum moment, Span 3:

$$28\,400 \times 12.9 - 2\,200 \times \frac{12.9^2}{2} = 184\,000 \text{ ft-lb.}; \quad w\,l^2 = 8 \times 184\,000 = 1\,472\,000 \text{ ft-lb.}$$

For Span 2:

$$w\,l^2 = 2\,200 \times 30^2 = 1\,980\,000 \text{ ft-lb.}$$

First, choose a beam section adequate for the maximum end shear which occurs in the center span (Span 2). For a shear of 33 000 lb. make the beam 12 by 30 in., giving

$$v = \frac{33\,000}{0.88 \times 12 \times 28} = 112 \text{ lb. per sq. in.} \quad M\,R = \frac{118.8}{12}\,b\,d^2 = -93\,100 \text{ ft-lb.}$$

for the stem, without using any compression steel. This moment requires a tension steel area, $A_s = \frac{M \text{ ft-lb.}}{1\,460\,d} = \frac{93\,100}{1\,460 \times 28} = 2.27$ sq. in. Use three 1-in. round bars.

This section will be accepted for all spans, making all end moments equal to 93 000 ft-lb., and the corresponding center moment for each span will be found, using the moment coefficients given in Table 2.

End Span 1.—For the end moment,

$$M_e = \frac{w\,l^2}{m}, \text{ or } 93\,000 \text{ ft-lb.} = \frac{894\,000}{m}$$

From this, $m = 9.65$. For this value of m for M_e, the value of m for M_c will be 9.75. Hence, $M_c = \frac{894\,000}{9.75} = 92\,500$ ft-lb. Again, using Table 2, c = 0.172 $\times$ 18 = 3.1 ft. Since M_c is less than M_e, no T is required at the center. $A_s = \frac{92\,500}{1\,460 \times 28} = 2.26$ sq. in. Use three 1-in. round bars. Of these three 1-in. round bars, make one straight (full length); one bent for shear; and one bent for double continuity at the column end of the beam.

Intermediate Span 2.—

$$M_e = \frac{w\,l^2}{m} = \frac{1\,980\,000}{21.3} = 93\,000 \text{ ft-lb.}$$

Thus, for M_e, m is 21.3; for M_c, it will be 9.7. Hence, $M_c = \frac{1\,980\,000}{9.7}$ = 204 000 ft-lb. From Table 2, c = 0.085 $\times$ 30 = 2.55 ft. The effective width of the 4-in. flange $= \frac{204\,000 - 93\,000}{5\,250^*} + 12 = 33$ in. $A_s = 5.00$ sq. in. Use two 1-in. square bars straight (full length); two 1-in. round bars bent for shear; and two 1-in. round bars bent for double continuity at each end of the beam.

End Span 3.—

$$M_e = \frac{w\,l^2}{m} = \frac{1\,472\,000}{15.85} = 93\,000 \text{ ft-lb.}$$

Thus, for M_e, m is 15.85; for M_c, it will be 8.33. Hence, $M_c = \frac{1\,472\,000}{8.33}$ = 177 000 ft-lb. From Table 2, c = 0.105 $\times$ 24 = 2.5 ft. The effective width of the 4-in. flange $= \frac{177\,000 - 93\,000}{5\,250^*} + 12 = 28$ in. $A_s = 4.33$ sq. in. Use two 1-in. square bars straight (full length); two 1-in. round bars bent for shear; and one 1-in. round bar bent for double continuity at the column end of the beam.

The bars bent for double continuity are bent up at the point of contraflexure at 45° and extend into the adjacent spans 1 ft. beyond the respective points of contraflexure nearest the column support. It will be seen that the

* 5 250 is the *MR* for 1-in. width of a 4-in. flange on a beam of 28-in. effective depth, which may be found in standard tables.

foregoing arrangement of bent bars furnishes three 1-in. round bars over the top of each support as required for the end moments of 93 000 ft-lb. The bars bent for shear are bent up at 45° next to the columns and at the outer ends. Likewise, they are hooked into the columns and at the outer ends.

No extra bars are thus required for the negative end moments as the necessary steel was obtained by proper bending and careful selection of bar sizes to make the steel areas correspond with the computed areas at all beam sections. Also, the end moments on each side of each column being equal, there will be no moment transmitted to the columns except a very nominal amount which may result from partial live loads on any of the spans.

In the foregoing example, say, 2.0 sq. in. of compression steel might have been utilized by extending the bottom straight bars into the columns and adjacent spans, making the negative end moments 129 000 ft-lb. instead of the 93 000 ft-lb. furnished by the concrete stem only. This would have resulted in somewhat smaller center moments and possibly a slight economy in steel.

If, for any reason, the end moments of an interior span are not made equal, a condition which is governed by the lengths and loadings of the adjacent spans, then the center moment is based on the mean value of the two unequal end moments. Similarly, if a certain end moment is assigned at Column 1 or Column 4, to be transmitted from the end spans to the end columns, then such end spans are designed for center moments based on the average end moments of these spans, the same as for interior spans with unequal end moments.

The shear reinforcement still to be provided in the form of stirrups is computed on the assumption that the concrete is permitted to take 40 lb. of shear per sq. in., and the bent shear bars are good for the added shear increment between the point of the first bend and the end of the span.

The gross end shear in the center span (Span 2) is 33 000 lb.; the stem area then takes $40\ b\ j\ d = 11\,760$ lb. and the bent bars will carry $3 \times 2\,200 = 6\,600$ lb., leaving a net shear, $V = 14\,640$ lb., to be taken by stirrups.

The minimum stirrup spacing is now found from the formula, $s = \dfrac{f_s\ A_s\ j\ d}{\text{net } V}$,

in which, $f_s = 16\,000$ lb. per sq. in; $j = 0.88$; $d =$ effective depth of beam; and $A_s =$ area of one stirrup $= 0.22$ sq. in. for $\frac{3}{8}$-in. round, 0.39 sq. in. for $\frac{1}{2}$-in. round, and 0.61 sq. in. for $\frac{5}{8}$-in. round U-stirrups.

Hence, for Span 2:

$$s = \frac{3\,080\ d}{V} \text{ for } \tfrac{3}{8}\text{-in. round U-stirrups} = 6 \text{ in.}$$

$$s = \frac{5\,500\ d}{V} \text{ for } \tfrac{1}{2}\text{-in. round U-stirrups} = 10 \text{ in.}$$

$$s = \frac{8\,550\ d}{V} \text{ for } \tfrac{5}{8}\text{-in. round U-stirrups} = 16 \text{ in.}$$

Any one of these three sizes may be chosen, but the $\frac{1}{2}$-in. round bar seems preferable for the 30-in. beam at a minimum spacing of 10 in., which is the

spacing required at the point where the first bar is bent up for shear. This spacing may be increased toward the center of the span to a point where the unit shear is only 40 lb., and the total shear is 40 $b\,j\,d$ = 11 760 lb., beyond which no stirrups are required. This gives a distance out from each end of $\frac{33\,000 - 11\,760}{2\,200}$ = 10 ft., where stirrups are required, or 30 — 2 × 10 = 10 ft. centrally, where no stirrups are required. The stirrup spacings for Spans 1 and 3 are determined in a similar manner.

In presenting this example sufficient descriptive matter was deemed necessary for the benefit of the reader. In actual practice, however, the complete numerical work could be reduced to about six lines for the 3-span problem. The outline and table herein given, together with the three other tables mentioned, furnish a complete equipment for the design of any concrete beam or slab in building construction.

The author's suggestion of inverting stirrups over the range of negative moments would not avail much since the stirrup ends are usually hooked over the top bars and some straight bars always extend the full length of the beam. Furthermore, the shear acts in the same direction whether over the range of positive or negative moments. The writer favors the use of stirrups merely for the purpose of supplying a deficiency in shear reinforcement wherever needed, and would utilize bent bars to the fullest extent as far as they are available in any given case.

The practical method of designing reinforced concrete beams herein given, deserves the careful consideration of structural engineers, and it is hoped that it may gain in favor to the extent of being incorporated into the building codes of other larger cities.

L. J. Larson,* Assoc. M. Am. Soc. C. E. (by letter).—The problem of properly distributing the steel in a concrete beam is well worthy of consideration. The difficulty of designing a rectangular beam to carry the relative large moments at the supports when a T-section is available to carry the smaller moments at the center of the span, makes any logical method of reducing the moments at the supports especially welcome. The author's procedure is indeed convenient and if the partial continuity of a beam is due to some freedom of rotation at its ends his conclusions are justified.

In many beams, however, the ends are fixed and the tangents to the elastic curves remain horizontal at those points. Such is certainly the case for the types of beams given by the author as fixed.

If the ends of the beams are fixed the division of moment between the center and the end of the span depends on the variation of the moment of inertia of the beam and not on assumptions. The location of the points of inflection for various distributions of the moments is as indicated in the author's Fig. 2. However, the location of the points of inflection and the true distribution of the moment depend on the variation of the moment of inertia of the beam. Let I_s be the moment of inertia of the beam for the section from the support to the point of inflection and let I_c be the moment of inertia of the

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central part of the beam, then the ratio, $\frac{I_s}{I_c}$, required to produce points of inflection at a distance, $k\,l$, from the support is shown in Table 3. The values of k lie between 0 and 0.5. The moment coefficients at the supports and at the center of the span are also given.

In Fig. 19 the ratio, $\frac{I_s}{I_c}$, and the moment coefficients are plotted against the values of k. For a rectangular beam having the same amount of steel at the center as at the supports the ratio, $\frac{I_s}{I_c}$, equals 1.0, and the moment coefficients agree with the accepted values for a fixed beam. For beam and slab construction the center is a T-beam and the ends are rectangular beams. The moment of inertia of the center is greater than that of the end, and the bending moment at the ends of the beam will be reduced below the usual value for a fixed beam.

TABLE 3.

	Values of k.							
	0	0.10	0.15	0.20	0.25	0.30	0.40	0.50
Ratio, $\frac{I_s}{I_c}$	0	0.103	0.315	0.817	2.00	5.12	271	.....
Moment at support	0	0.045	0.064	0.080	0.093	0.105	0.120	0.125
Moment at center	0.125	0.080	0.061	0.045	0.032	0.020	0.005	0

To what extent the moments and stresses will be affected by the variation in the moment of inertia can best be shown by a specific example. Consider a T-beam having a flange 50 in. wide and 4 in. thick and a stem 10 in. wide. The total depth of beam and slab is 22 in. (to the steel). Assume 4 sq. in. of steel. The moment of inertia of such a beam is 18 570 in.^{4} At the support consider a rectangular beam 10 in. wide and 22 in. deep (to steel) with the same amount of steel. The moment of inertia of this beam is 11 550 in.^{4}

Using the notation in Fig. 19, $\frac{I_s}{I_c} = 0.62$, the bending moment at the support is $0.076\,w\,l^2$, that at the center of the beam is $0.049\,w\,l^2$, and the resulting stresses are $0.000071\,w\,l^2$ and $0.000041\,w\,l^2$ at the support and at the center of the beam, respectively. Thus, the stress at the support is about 70% higher than at the center of the beam if the areas of tension steel at the two points are equal. If the area of the steel at the support is reduced to 2.0 sq. in., or one-half that at the center of the beam, the ratio, $\frac{I_s}{I_c}$, is 0.40, the moment at the support equals $0.07\,w\,l^2$, the moment at the center of the span equals $0.055\,w\,l^2$, the stress at the support is $0.000122\,w\,l^2$,

and the stress at the center is $0.000046\ w\ l^2$. Hence, the steel at the support will be stressed about two and one-half times as high as that at the center of the span. Any further reduction of the steel at the support aggravates the situation. Simply reducing the steel at the support does not reduce the moment a corresponding amount.

If the amount of the steel at the support is much below that required, it must be over-stressed, or the steel at the center of the span will be subjected to less than the working stress. Of course, a re-adjustment will take place before the structure collapses. Most statically indeterminate structures have the happy faculty of shifting the load from the weak or over-stressed parts to those less stressed. However, this re-adjustment cannot take place until some failure or yielding occurs. In the case of a concrete beam the concrete may flow, the steel may pass the yield point, or the rods may slip at the supports. One of these failures must occur before the center of the beam can receive a greater proportion of the moment. To what extent such a yielding at the supports may be tolerated, must be determined from practical consideration.

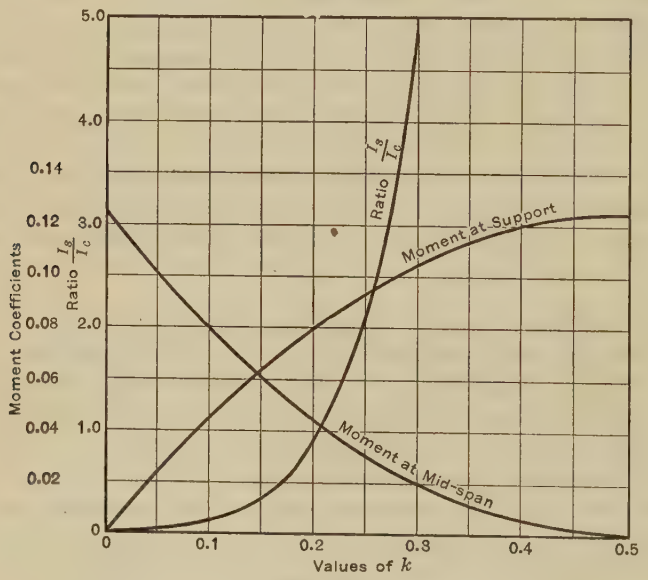

FIG. 19.

Shear Reinforcement.—Although it may be more convenient to use vertical stirrups as shear reinforcement, it is possible to make bent-up bars quite effective. Since some bars are generally bent up in continuous beams to help carry the negative moment they may very well be used as shear reinforcement. Some tests made by the writer at the University of Illinois show the effectiveness of such bars.

The beams tested were 8 in. wide, 17 in. deep, and 18 ft. long; d was 16 in. They were supported on a span of 12 ft., with an overhang of 3 ft. at each

end. Equal loads were placed at the third points of the central span and 2 ft. 8 in. outside each support. This loading produced a moment at the supports twice that at the middle of the beam. About 2% of steel was provided at the supports. The bars were bent up at angles of 22½, 30, and 45° at various spacings up to 16 in. The distance of the first bend from the support varied from 8 to 24 in. In all, forty beams were tested. Of this series practically all the beams developed average vertical shearing stresses of more than 400 lb. per sq. in. and the failure was seldom caused by diagonal tension. Judging from these tests it appears that in beams having bent-up bars the amount of stirrup steel may often be materially reduced if such bars are considered as shear reinforcement.

Anton Brandtzaeg,* Assoc. M. Am. Soc. C. E. (by letter).—The distribution of the steel should agree with the distribution of the stresses. When taken in its broadest aspects, the determination of the stresses set up in an actual structure by any given condition of loading, usually should involve the consideration of two different questions: (1) What stresses would exist if the same structure were built of an ideal, homogeneous, isotropic, and elastic material; and (2) in what manner does the material actually to be used for the structure deviate from the ideal, and how will this deviation influence the distribution of the stresses?

The first question involves a problem in mechanics. Generally, a definite solution of the problem may be found; there is no need of resorting to arbitrary assumptions, although it is necessary for convenience to have simple, standard formulas to be used for standard cases.

Thus far, the discussion has touched but little on the second of the questions mentioned. Mr. Myers' paper contains, however, some far-reaching statements regarding this point. From the "General Principles" stated, and the conclusions drawn from them, it would seem that the author considers the non-homogeneity of the material to have such a large influence on the distribution of moments in continuous beams of reinforced concrete, that the theory of elastic beams is rendered fairly useless as a basis for computing these moments. The question thereby raised—as to what extent the distribution of moments along a continuous beam of reinforced concrete will follow the laws of flexure of elastic beams—is one that has to be answered before the establishment of a sound basis for the design of such beams will be completed.

Tests of continuous structures of reinforced concrete are laborious and expensive; nevertheless some reports of such tests are to be found in current engineering literature. It may not be out of place in this discussion to refer briefly to a few such tests, as they throw some light on the problem in general, as well as on some of the particular recommendations contained in the paper.

Among tests made in the United States, those carried out by Dr. Mikishi Abe at the University of Illinois on a large number of rigid frames of reinforced concrete, are well known.† The general result of the tests was that a

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† "Analysis and Tests of Rigidly Connected Reinforced Concrete Frames," by Dr. Mikishi Abe, *Bulletin No. 107*, Univ. of Illinois, 1918.

fair agreement was found between the stresses computed on the assumption of elasticity and those found in the tests by strain-gauge measurements.

Among tests made in Germany there are some which attack in a very direct way the problem of moments in continuous beams. As early as 1907 the construction firm Wayss and Freytag, Incorporated, undertook the testing of three large T-beams, continuous over two equal spans of about 19 ft. 4 in. clear width.* Naturally the method used in these early tests was not far enough developed to give very complete results, and the tests now have mainly historical interest, although they demonstrated that the beams must have acted very much like continuous beams of elastic material.

A comprehensive series of tests of continuous T-beams was made by Scheit and Probst, first reported in 1912.† Professor Probst describes the tests in his book on reinforced concrete.‡ A description of the tests is also given in a recent book by Taylor, Thompson and Smulski;§ a general description of the tests is therefore hardly warranted.

The test data give detailed information on the loads applied, the deflection of the beams, and the rotation of the beams over the supports. Professor Probst compares the relative deflections of the beams under several conditions of loading and support with those computed for elastic beams of uniform cross-section under the same conditions. He finds fair agreement, and concludes that continuous beams of reinforced concrete, freely supported, in general follow the same laws as similar elastic beams. He finds that for beams poured monolithically with the columns, the effect of the columns on the bending moments is very pronounced.

Few strain measurements were made in these tests, and little is known of the internal stresses actually existing in the beams. All the beams failed by tension in the steel, and a rough estimate of the stresses at failure is made on the basis of the elastic strength of the steel as determined by auxiliary tests. As the steel evidently was not of a very mild grade, even this estimate is rendered rather uncertain. Nevertheless, a sample of the results may be worth mentioning in connection with the present discussion.

Two identical beams, continuous over two spans, both of which were loaded, failed by tension in the steel over the support. An estimate on the basis of the yield-point stress for the steel gives a negative moment over the support at failure of slightly less than $\frac{W\,l}{10}$. For a similar elastic beam of uniform cross-section the moment would be nearly $\frac{W\,l}{8}$. (The load was only approximately uniformly distributed.) These tests indicated, therefore, that the moment over the support was less than that computed for elastic beams. It is interest-

* Professor E. Moersch, who planned the tests, describes them in his book, "Der Eisenbetonbau." Fifth Edition, Stuttgart. 1920, Vol. 1, second half, pp. 318*ff*.

† "Untersuchungen an durchlaufenden Eisenbetonkonstruktionen," by H. Scheit and E. Probst, pub. by J. Springer, Berlin, 1912.

‡ "Vorlesungen über Eisenbeton," by Professor Dr. E. Probst, pub. by J. Springer, Berlin, 1917, Vol. 1, pp. 442 to 495.

§ "Concrete Plain and Reinforced," by F. W. Taylor, S. E. Thompson, and E. Smulski, John Wiley & Sons, N. Y., 1925, Vol. 1, pp. 64 to 68.

ing to consider this beam in relation to the views expressed by the author. The beam had the same area of steel over the support and at the center of the spans. Suppose that the designer of such a beam had decided to assume the moments at the center of the span and at the support to be the same, and had used as the governing condition that the sum of the positive and the average of the negative moments should be $\frac{W\,l}{8}$. He would then have distributed the steel as it was in this beam and would have designed it for $\frac{W\,l}{12}$ at the support. Actually, the steel stress corresponded to a negative moment of about $\frac{W\,l}{10}$, and the beam would have been about 15% too weak for the load for which it was designed. The author's recommendation for beams over end spans is, however, that they should be designed for $\frac{W\,l}{10}$ at the center and $\frac{W\,l}{20}$ at the interior support. The beams referred to had twice as much steel at the support as recommended by the author; the fact that they failed at that point and not in the center must indicate that they had too little, rather than too much, steel there.

The author's statement that "an interior beam framing into a column at each end, in line with similar beams in the adjacent spans, unquestionably is fixed at its supports", does not agree with the results from some tests of beams over three equal spans, resting on concrete columns, which were included in this series. The two end spans only were loaded. Large rotations were observed at the interior supports, and large cracks developed in the top of the beam along the unloaded middle span. It may also be noted that very large cracks developed in the columns themselves.

Another series of tests of considerable interest is that carried out by Bach and Graf in Stuttgart, reported in 1920.* The tests were made under the auspices of the German Committee on Reinforced Concrete. The plans for the tests were proposed by Professor E. Moersch, who has included a description of the tests in his book.†

The specimens for these tests were of rectangular cross-section, in most cases they were about 11¾ in. wide and 7⅞ in. deep. The beams rested on two supports, about 9 ft. 10 in. apart; they were extended as cantilevers about 4 ft. past the supports, both ways in some tests, one way in others. Equal loads were applied at six points uniformly spaced between the supports; in addition, there was one load at each cantilever end of the beam, about 3 ft. 3⅝ in. outside the supports. These latter loads were applied by separate mechanisms, and were regulated throughout the tests so that the cross-sections of the beams immediately over the supports remained in their original vertical position. The beams thus were actually fixed at one or both of the

* "Versuche mit eingespannten Eisenbetonbalken," by Dr. Ing. C. Bach and O. Graf, Heft 45, Deutschen Ausschusses für Eisenbeton, Berlin, 1920.

† "Der Eisenbetonbau," by Professor E. Moersch, Fifth Edition, Vol. 1, second half, pp. 310–318.

supports; the magnitude of the cantilever loads required to maintain this condition gave direct measurements of the negative moments.

Three sets of tests, each including three identical specimens with cantilevers at both ends, will be mentioned here. In two of these sets the steel was distributed approximately according to the distribution of moments in an elastic beam of uniform cross-section, fixed at both ends. The quantities of steel in one set were 0.3% at the center and 0.63% over the supports; in the second set the percentages were 0.77 and 1.63, respectively. In both these sets the negative moment, as actually measured, agreed closely with that computed for elastic beams, throughout all the stages of loading. The measured moment slightly exceeded the computed one at the higher loads. Cracks developed at about the same rate at the center and over the supports, and the final failure occurred through tension in the steel, in some cases at the center, in others at one of the supports. Professor Moersch takes this as an indication that the steel in these beams had been properly distributed.

In the third set of tests the distribution of the steel was the reverse of that in the second set, the quantity at the center being 1.63%, and that over the supports 0.77 per cent. Special interest is attached to this set of tests in connection with the present discussion, since the distribution was very nearly that advocated by the author for interior spans of continuous beams. At early loads the negative moment in this set, as in the previous ones, followed the values computed for similar elastic beams of uniform cross-section. After cracking had started over the supports, the observed negative moment was smaller than the computed one, the difference increasing as the test proceeded. However, the difference between the computed and the actual moment was not as great by far as if the moments had been distributed proportionately with the steel. To take an example: At a total load between supports of about 22 000 lb. the negative moment for a similar fixed elastic beam of uniform cross-section would be about 18 300 ft-lb. The moment measured was about 14 400 ft-lb., or 79% of the "theoretical" value. Taking the moment at the center of a similar beam simply supported, and assuming it to be distributed between the center and the support proportionately with the steel, one computes a negative moment of about 8 700 ft-lb., or 47.5% of the "theoretical" value. The steel stresses at this load, computed from the observed moments by ordinary formulas, amounted to about 16 500 lb. per sq. in. at the center and 40 000 lb. per sq. in. at the supports. The yield point for this steel was at about 43 000 lb. per sq. in. As the load was raised, the increase in the negative moment soon became very slow; the steel over the supports yielded, with cracks opening up, and the further increase in load had to be carried by the steel at the center. Ultimately, the beams of this set carried about the same load as those of the preceding one.

These tests show that the designer may distribute the steel according to the recommendations of the author only if he is prepared to accept the certainty of premature yielding of the steel over the supports, with the resulting large cracks. Until such yielding occurs the moments will not follow an arbitrary distribution of the steel, even if they do deviate somewhat

from the "theoretical" values. Certainly the foregoing steel stresses do not indicate that the material in these beams had been distributed to the best advantage.

The tests here mentioned demonstrate the advisability of showing caution in basing the distribution of the steel in continuous beams on too loose assumptions. They indicate in a general way that a computation based on the laws of flexure of homogeneous beams of uniform cross-section is likely to give for the bending moments values that are reasonably close to those actually existing in the beams. They do not, however, point to any definite conclusions as to just in what manner the action of continuous beams of reinforced concrete deviates from that of homogeneous beams.

The great difference between a homogeneous beam and one of reinforced concrete in which "hair cracks" have developed on the tension side of the beam, is very obvious. From a general consideration of the case it might seem that, in order to obtain correct values of the moments, one should neglect the concrete on the tension side of the neutral axis in computing the moments of inertia, and introduce different moments of inertia, according to the different amounts of steel and shapes of the concrete section at the center and over the support, into the ordinary formulas for moments in continuous, elastic beams. In fact, other discussors have explicitly assumed this to be so. The tests mentioned do not seem to indicate that the moments computed in this manner will, on the whole, be more correct than those computed on the assumption that the moment of inertia is uniform throughout the beam. The beams over two equal spans, tested by Probst, had the same amount of steel at the center and over the support. The beams were of a T-section, with a slight amount of compressive steel at the support, and the ratio, $\frac{I_p}{I_n}$, for these beams was about 1.08, neglecting tension in the concrete. According to this, the negative moment should have been only slightly less than that computed for a beam of uniform cross-section. However, the test indicated a negative moment as much as 20% less. Most of the beams tested by Bach and Graf had twice as much steel at the supports as at the center. The ratio, $\frac{I_p}{I_n}$, for the second of the three sets of beams was about 0.61. Accordingly, the negative moment should have been about 10% higher than that computed for a beam of uniform cross-section. However, the negative moment observed exceeded only by about 3% the moment computed for such a beam. For the extreme case of the third set of beams, for which $\frac{I_p}{I_n} = 1.63$, by computation the negative moment should be 87% of that for a beam of uniform moment of inertia, while the observed moment was 79 per cent.

Professor Moersch,* gives an interesting discussion of the question as to what sections should be considered in computing the moments of inertia of concrete beams for the purpose of determining moments in continuous beams.

* "Der Eisenbetonbau," Fifth Edition, Vol. 1, second half, pp. 284–310.

The discussion is based on the results of a number of tests of simply supported beams, some of which had rectangular cross-section, while some were T-beams, and some had an "inverted T-section" with the steel at the same side of the beam as the slab. The latter beams represented the conditions at the supports of a continuous T-beam. In all these tests the relations between bending moments and deformations of a short length of beam were directly determined.

Professor Moersch applies these observed relations in a semi-analytical study of the action of several types of beams with one or both ends fixed. He assumes in each case a beam in which the steel has been distributed according to the distribution of moments in a similar beam of uniform cross-section. The dimensions of the beam and the reinforcement have been chosen so that the cross-section at every point along the beam is equal to that of some one of the simple beams tested. By means of the observed relations between moments and deformations of short lengths of beam, he determines what total deformations of the assumed beams would correspond to the distribution of moments considered. He treats in this manner a rectangular beam fixed at both ends, a rectangular beam fixed at one end, and a T-beam fixed at one end. He finds in all cases that the total deformations of the beams, determined on this basis, agree very nearly with the conditions of support in each case; in other words, that the distribution of bending moments used is very nearly right. Professor Moersch draws from this the general conclusion that if the steel in a concrete beam is distributed in accordance with the bending moments computed for a similar elastic and homogeneous beam of cross-section equal to the full concrete section, then the actual bending moments will correspond very nearly to the moments thus computed. He refers to the tests by Bach and Graf, previously mentioned, as supporting this conclusion.

The tests seem to indicate that in many cases of great practical importance this statement by Moersch may be nearly right. It is obvious that if this is so, it is due to a rather complex set of reasons, and it would seem that there must be limitations to the applicability of the rule.

Among the possible sources of such limitations, there is one which was not considered in the analysis by Moersch, and which is hardly capable of being fully allowed for analytically, namely, the danger of slipping of bars over the supports of a continuous beam. The writer is convinced that serious limitations on the conformity of the actual bending moments in continuous beams of reinforced concrete with the computed ones, may arise in many cases from this source. There are two reasons why the danger of slipping of bars is probably greater at the supports of a continuous beam than at most other points of a reinforced concrete structure. In the first place, the maximum bond stresses occur at the supports. At the same time the steel is there stressed so highly that hair cracks in the concrete are almost certain to develop even at working loads. It was amply demonstrated by the tests made by Professor Abrams at the University of Illinois,* that when hair cracks develop

* "Test of Bond Between Concrete and Steel," by Duff A. Abrams, M. Am. Soc. C. E., *Bulletin No. 71*, Univ. of Illinois, 1913.

along any part of a beam, the resulting "anti-stretch slip" impairs greatly the ability of the beam to resist the "regular" bond stresses along that part. In the second place, in most cases the steel in such a beam is firmly supported on the forms during the pouring of the concrete and is prevented from settling together with the concrete. As the settlement close to the top of the beam will be greater than that close to the bottom, the detrimental effect of the settlement on the bond strength will also be more pronounced at the top. That such settlement greatly affects the bond strength, was also shown by Professor Abrams' tests.* It should be noted in this connection that in the German tests, smaller bars were used than would ordinarily have been used in similar beams in American practice. Nevertheless, it may be that slipping of bars may have occurred and this may be responsible in part for the small value of the negative moment in the beams over two spans, as tested by Probst. The fact that cracks suddenly opened a short distance from the support some time before the maximum load was reached,† may possibly indicate that this was the case.

On the basis of the tests referred to it would seem that the designer can hardly do better than to compute the bending moments in a continuous beam of reinforced concrete as if he were dealing with an homogeneous beam of the same cross-section as the concrete beam. The moments thus computed are likely to be reasonably close to the actual ones. It is doubtful, at best, whether the introduction of varying moments of inertia according to the varying amounts of steel will give results that are more correct than those obtained by the simpler method. The designer should keep in mind, however, that he is dealing with a non-homogeneous material, and that in particular the bond between the concrete and the steel may in many cases be insufficient to maintain the conditions on which the computation is based. This consideration may often make it advisable to provide some extra steel at the center of the beam, in order that the beam may be strong enough there, even if some slipping of bars should occur at the supports.

The possibility that slipping of the bars may act to reduce the negative moment at the support, however, should not be considered as a reason for reducing the steel at the supports. That would only result in increasing, not only the tension in the steel, but also the bond stresses. The ultimate result of any large reduction of the steel at the supports from the amount required for the computed moment, will be premature yielding of the steel, or bad slipping, either of which will cause cracks to open. It will be seen that the danger of slipping of the bars over the supports furnishes one reason, in addition to the necessity of providing for an unequal distribution of loading, why the sum of the positive and the negative bending moments as given in the standard formulas of building codes, should be greater than $\frac{W\,l}{8}$.

There seems to be need of further experimental study of the question as to how the distribution of moments in continuous beams of reinforced con-

* "Test of Bond Between Concrete and Steel," by Duff A. Abrams, M. Am. Soc. C. E., *Bulletin No. 71*, Univ. of Illinois, 1913, p. 105.

† "Vorlesungen über Eisenbeton," by E. Probst, 1917, Vol. 1, p. 459.

crete deviates from that in homogeneous elastic beams. Such experiments would doubtless be of the greatest value if they could give information not only as to the deviations found in certain cases, but also as to the causes underlying these deviations.

JOHN J. GOULD,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has shown certain methods of designing reinforced concrete beams and slabs. He has failed, however, to substantiate these methods by actual tests on beams, or by scientific reasoning; nor does he justify in any way his rejection of the Theorem of Three Moments in designing reinforced concrete structures.

Generally speaking, the author believes that by placing reinforcing steel in a certain way he is able to regulate exterior moments and working stresses; that the distribution of the steel—a part to the center and the remainder to the support, or *vice versa*—is entirely optional with the designer.

In Fig. 20 is shown what the deflection line of a continuous beam would be if all the steel were placed in the top and the continuous beam were designed as a series of cantilevers. Besides the questionable stability of the system, cracks in the middle of the spans would occur. Fig. 20 leaves no doubt about the author's erroneous fundamental assumptions in designing reinforced concrete structures.

According to the author's "General Principles", T-beams "cannot readily be analyzed by the three-moment theorem or similar theory". The writer believes that the extensive tests of continuous reinforced concrete beams, made in Germany by Dr. E. Probst† and other engineers, may dissipate some of the existing doubt as to whether or not the Theorem of Three Moments should be applied to reinforced concrete structures

These tests of T-beam sections show how little influence all errors make on exterior moments. They were made under the assumptions of constant moment of inertia, straight line variation of concrete stresses, and neglect of shearing stresses.

In all these tests the theoretical moments of continuous beams were calculated on the basis of the Theorem of Three Moments. These moments were then compared with the moments calculated from measured deformations of the concrete and steel after the beams had failed from ultimate loads. The sections of all beams were of the same shape and dimensions. The reinforcing bars were located according to the theoretical moments and shears.

Test Type I, of T-beams on two supports, 3.0-m. span, was made for comparison with other continuous beams. In Test Type II, of continuous T-beams on three supports with two equal 3.0-m. spans, the failures occurred over the interior supports, which indicates that the maximum moment of the beam is negative. The difference of 20% between the theoretical moment over the interior support and the moment calculated from actual stresses is because the loads were not uniformly distributed and a possible settlement of the support may have occurred.

* Structural Engr., L. H. Nishkian, San Francisco, Calif.

† "Vorlesungen über Eisenbeton," Bd. I, Berlin, 1917.

The comparison of relative deflections between Test Types I and II in the center of the beams gave a theoretical relationship, $R\,e$, of,

$$R\,e = \frac{D\,e\ \text{II}}{D\,e\ \text{I}} = \frac{P\,L^3}{192\,E\,I} \div \frac{5\,P\,L^3}{384\,E\,I} = 0.40$$

in which, $D\,e$ I is the deflection of the beam, Test Type I; similarly, for $D\,e$ II, $D\,e$ III, etc. By comparison the actual measured deflections show the following relationship:

	Before cracks occurred.	After the first cracks occurred.	At approximately one-half the ultimate load.
Loads, in tons per meter =	3.6	6.6	10.4
$R\,e = \frac{D\,e\,\text{II}}{D\,e\,\text{I}}$ =	0.90	0.42	0.45

In Test Type III, a continuous T-beam on four supports, three equal, 3.0-m. spans, the theoretical moments show very close relationship with the moments calculated from measured deformations.

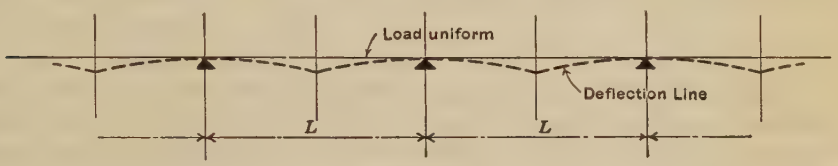

FIG. 20.

The comparison of relative deflection between Test Types I and III for the end spans (the middle span was not loaded) gives the following relationship:

$$R\,e = \frac{D\,e\ \text{III}}{D\,e\ \text{I}} = 0.260\,\frac{P}{E\,I} \div \frac{5\,P\,L^3}{384\,E\,I} = 0.74$$

while the comparison of the actual measured deflections show a relationship of:

Loads, in tons per meter =	3.6	6.6	10.4
$R\,e = \frac{D\,e\ \text{III}}{D\,e\ \text{I}}$ =	0.69	0.63	0.69

In Test Type IV, the construction is similar to Test Type III. The beams, however, were built integral with the supports, that is, a typical reinforced concrete structure. The deflections were found to be smaller than those in Test Type III. The influence of the columns on the beam moments were about 20% $\left(\text{for Test Type III, } M = \frac{1}{9.7}\ W\,L\text{; and for Test Type IV, } M = \frac{1}{12}\ W\,L\right)$.

The measured relative deflections between Test Types I and IV are:

Loads, in tons per meter =	3.6	6.6	10.4
$R\,e = \frac{D\,e\ \text{IV}}{D\,e\ \text{I}}$ =	0.36	0.37	0.34

The comparison between these coefficients and the coefficients found for Test Type III shows that the columns fix the beams to a considerable degree. If the beams were entirely restrained at their ends, these coefficients would become 0.20.

In an ordinary reinforced structure the writer would recommend considering all beams as being entirely restrained at outside columns for lower stories, while for upper stories the moment of inertia and the rigidities should be determined with the following considerations.

Take the moment of inertia of the columns about the neutral axis. The rigidity of the columns would then be the combined rigidity of the column above and below the floor-beam under consideration. An analysis on this basis would result in a bending moment for the beam at the outside support with an average value of $\frac{WL}{12}$ to $\frac{WL}{16}$.

Test Type V, was a continuous beam on six supports, with spans of 3.0, 2.5, 4.0, 2.5, and 3.0-m., the 3.0 and 4.0-m. spans only being loaded. The comparison of relative measured deflections between Test Types I and V in the center of the 3.0-m. spans, gives a relationship of:

Loads, in tons per meter	= 3.6	6.6	10.4
$Re = \frac{De\,V}{De\,I}$	= 0.80	0.67	0.72

These coefficients are very similar to those found in Test Type III. The theoretical exterior moments compare closely with the moments calculated from actual stresses. The continuous effect of five spans, therefore, should be considered.

Conclusions.—Dr. Probst's tests of continuous beams have shown that:

(1) Moments computed on the basis of the Theorem of Three Moments check within a few per cent. with the moments calculated from actual stresses.

(2) The measured relative deflections of a simply supported reinforced concrete T-beam and continuous T-beams are almost identical with the theoretical relative deflections of beams of homogeneous material and of constant moment of inertia.

(3) A few small cracks occurred at the points of zero moments, thereby locating these points at positions agreeing with the Theorem of Three Moments.

(4) In curvature, the deflection line of reinforced concrete T-beams resembles that of beams of homogeneous material and of constant moment of inertia.

(5) Continuous reinforced concrete beams, therefore, should be designed in accordance with the laws of the Theorem of Three Moments.

(6) The designer should take into consideration the end restraint of beams which frame into columns.

(7) Arbitrary coefficients for computing reinforced concrete structures are irrational, and in very many cases contradict the Theorem of Three Moments.

Boyd S. Myers,* M. Am. Soc. C. E. (by letter).—The discussion shows a wide diversity of opinion on this very important subject.

The purpose of the paper was to lead to a definite approval or rejection of the principles discussed. If those principles are correct, they should be used

* Cons. Structural Engr. (Brown & Myers), Oklahoma City, Okla.

more generally by engineers on account of the economy of material secured; and if they are radically in error, evidently they should be discarded.

Hundreds and probably thousands of structures have been built in the United States, for which the designs were based on the assumption that the steel in the top over the supports need not exceed $\frac{W\,l^2}{24}$, or one-half the amount usually placed in the bottom at mid-span. These structures have given years of satisfactory service and show no visible signs of distress or weakness. This fact, however, does not necessarily indicate that the assumption is theoretically correct. Many of these designs were executed by men receiving their technical training in the best schools of the country, and their practical training with some large reinforcing bar company. Probably the reduction in the total tonnage of steel for a given project was one of the reasons for the assumption. Undoubtedly, if it is a radical departure from the true theory, there should be failures on record, with causes readily traceable to a weakness in the design. Faulty design has been the cause of relatively few failures of concrete structures.

The theoretical distribution of the moments in fully restrained concrete beams is well shown in Mr. Larson's discussion. If the net section of the beam is used in computing the moments of inertia, the concrete on the tension side of the neutral axis being ignored, the results should give the correct distribution of the moments, and, consequently, the correct distribution of the reinforcing steel. A balanced section of the beam should be used in computing the moments of inertia. A flange just wide enough to develop the tension steel should be used with both steel and concrete working at their maximum design stresses. The top steel at the supports will vary from 0.65 to 0.80 of that usually placed at mid-span for the ordinary case. Evidently, it would be more nearly correct to design interior spans for a moment of about $\frac{W\,l^2}{14}$ and to place the same area of steel in the top over the supports as at the bottom at mid-span.

The writer believes that the discussions have been of considerable value. He is fully convinced of the need of a comprehensive series of actual tests to link up in a better way theory and practice in concrete design.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1626

WATER-PROOF MASONRY DAMS*

By W. Watters Pagon,† M. Am. Soc. C. E.

With Discussion by Messrs. Karl R. Kennison, P. A. Beatty, R. W. Gausmann, H. F. Dunham, H. de B. Parsons, Alfred D. Flinn, J. B. W. Gardiner, T. Kennard Thomson, V. Bernard Siems, B. F. Jakobsen, Eugene E. Halmos, Edward Wegmann, Malcolm Elliott, K. E. Hilgard, Thomas H. Wiggin, Jerome W. Howe, W. H. R. Nimmo, E. W. Lane, Nathan C. Johnson, F. W. Scheidenhelm, and W. Watters Pagon.

Synopsis

Until recent years little attention was paid to the possibility that masonry dams might be liable to uplift forces of considerable magnitude, resulting in a dangerous reduction of their factors of safety. When general interest in the subject was aroused finally, the line of attack chosen, save in a few sporadic cases, was to neutralize the uplift by additional masonry. It is the purpose of this paper to draw attention to another method, which has been somewhat foreshadowed by recent designs; namely, that of preventing the uplift, rather than neutralizing it.

Briefly, the method consists in the provision of a water-tight membrane or sheet, placed as near the up-stream face of the dam as is practicable, with in addition a somewhat customary system of under-drainage to ensure the removal of any seepage from the foundation bed. A few suggestive general details are included to illustrate possible means of carrying out this general idea.

A brief history of dam design is given, showing the gradual development of knowledge concerning dams and the reasons for using the several forces that control the profile. As there is so little exact knowledge of the actual internal forces, no hard-and-fast rules can be laid down, but the best modern

* Presented at the meeting of November 3, 1926.
† Cons. Engr., Baltimore, Md.

practice is discussed and a set of formulas of the usual type is stated, actual designs being worked out to determine a specific result.

To determine the amount of saving actually obtainable in specific projects, two dams were designed, one low and one high. Alternate designs for each of these considered, first, that upward pressure would act; and, second, that all uplift would be eliminated in front of a membrane placed 3 ft. more or less from the water face. These designs are shown and the masonry quantities tabulated. Superimposed curves show the relative differences and the saving in masonry.

Brief History of Dam Design

The building of masonry dams has long been practiced, but their rational design has only been established in recent years. Many large masonry dams were built in Spain during the Middle Ages, but often such dams were subject to excessive stresses due to dead weight alone. Modern design dates from about 1853, when de Sazilly, a French engineer, deduced the first general equations for the profiles of dams. His designs were based on the principle of limitation as regards two internal elements: (1) The pressures in the masonry; and (2) the tendency toward shear, or sliding. He also proposed two conditions of exterior loading, namely, with reservoir full and reservoir empty. He limited his allowable pressures to about 6 tons per sq. ft., as compared with modern stresses up to 20 tons; hence his designs were excessively safe. He had observed that no dams had failed because of shear, and, therefore, considered that this did not enter the problem.

Succeeding him another French engineer, Delocre, carried the investigations further in the design of the Furens Dam. He devised a theoretical profile which would meet the same conditions.

Rankine introduced the next important idea. He evolved a theoretical profile that would meet the required pressures economically, limiting the lines of pressure to the middle third of the section. Moreover, since the line of pressure for the full head of water is sharply inclined, whereas that for the reservoir empty is nearly vertical, he proposed that the unit stress at the toe be restricted to a lower value than that at the heel. Similar profiles have since been devised.

Modern dam design may be said to date from the Quaker Bridge Dam of the New York Water Supply. This exceeded in height any existing dam by more than 100 ft. Edward Wegmann, M. Am. Soc. C. E., made a careful study of the design, using known principles to devise practical profiles by trial and error. He also increased the allowable pressure to 16 tons, based on a successful Spanish dam which had carried this stress for some centuries. With this increased allowable stress, the most important element then became the position of the line of pressure with reference to the middle third.

Up to this time only the weight of masonry and the horizontal pressure had been considered. It was recognized, however, that masonry is not impervious, but that leakage and damp spots indicated hydrostatic pressure within

the structure. Numerous failures emphasized the fact that something was wrong. In the Wachusett Dam uplift was allowed for; moreover the failure of the dam at Austin, Pa., was attributed to the neglect of uplift. The late Frederic P. Stearns, Past-President, Am. Soc. C. E., maintained that uplift should be considered over the entire area of section and at the full hydrostatic value. Mr. Wegmann still maintained that no uplift should be considered, and there were other opinions between these extremes.

In 1912, the late Charles L. Harrison, M. Am. Soc. C. E., proposed* three possible general conditions:

1.—Contact with a rock bed such as would preclude uplift, and no joints in the masonry.

2.—Porosity such that the water pressure would be at the full reservoir head at the heel and at tail-race head at the toe, varying uniformly for intermediate points.

3.—Full hydrostatic head at the heel and the head of the issuing stream at the toe.

After considerable discussion it was generally agreed that uplift must be considered, as had already been done in the Wachusett, Olive Bridge, and Kensico Dams. For these structures the uplift was assumed to vary linearly from two-thirds of the hydrostatic head at the heel to zero at the toe. This probably nearly represents present practice. Ice pressure has been variously considered as lying between the crushing strength of ice (47 000 lb. per sq. ft.) and zero. In recent years, the maximum intensity has been frequently used in northern climates.

Quantitative Effect of Uplift on the Size of Dam

A triangular cross-section is theoretically correct for economy, but cannot be entirely adopted in practice. Let, Δ = the ratio of the weight of masonry to that of water per unit volume; H = the depth of water, in feet (approximately the height of masonry above the section); and L = the width of base at the section. Imposing the requirement that the line of pressure follow the middle-third points, it is found that $L = \frac{H}{\sqrt{\Delta}}$ or, if $\Delta = \frac{7}{3}$, $L = 0.654\ H$, provided there is no uplift. When, however, provision is made for uplift by the two-thirds rule, there results, $L = \frac{H}{\sqrt{\Delta - \frac{2}{3}}}$, or $L = 0.775\ H$. For the two cases, the area of the vertical cross-section is $0.327\ H^2$, and $0.3875\ H^2$, respectively, the difference being 15.5% of the larger, or 18.5% of the smaller.

The comparison when ice thrust is considered is not so simple; but assuming a thrust at the top of the dam of 47 000 lb. per lin. ft., there results:

With pressure,

$$L = \sqrt{\frac{4\ 500 + H^2}{\frac{5}{3}}}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 142.

and without pressure,

$$L = \sqrt{\frac{4\,500 + H^2}{\frac{7}{3}}}$$

and the ratio of the respective lengths, or areas, is $\sqrt{\frac{5}{7}}$. The saving in masonry will be 15.5% of the larger as before. Hence, no further consideration need be given this factor.

The Water-Proofing Membrane

Apparently, John R. Freeman, Past-President, Am. Soc. C. E., was the first to propose the economy of water-proofing; he suggested* that a thin sheet of lead be built into the dam a few feet from the water face, thus absolutely preventing any uplift within the dam. His idea had reference to the proposed Housatonic Dam, but it was not used because the other engineers considered that the foundation conditions precluded the possibility of uplift, and they proposed that the masonry be so made as to be water-proof. Hence, the idea was dropped and almost forgotten.

At that time it was thought that masonry could always be made water-tight throughout, but later experience has shown the writer that tightness is by no means certain, so that serious spalling takes place on bridges and dams in cold climates and extensive lime deposits form on the surface in warmer climates, even on structures for which it would be presumed that the concrete was well made. Construction joints, also shrinkage and temperature cracks, may allow some leakage, and thus cause uplift, at least at those sections. Since, therefore, no important dam would be built nowadays without considering uplift, the whole question of water-proofing appears to be important.

Allen Hazen, M. Am. Soc. C. E., has described† a dam in Australia the back of which was built with special care to insure water-tightness. Porous tile pipes were placed parallel to and within a few feet of the back face leading down to a drainage gallery near the base. It was assumed that by collecting the water near the back no uplift could develop beyond that point. The foundation was excellent, but a cut-off wall was built to considerable depth. This case is the only one known to the writer in which provision was made for eliminating uplift and considered in the design.

In the Kensico and Olive Bridge Dams, tile drains and drainage galleries were provided, but were not considered as preventing uplift; in fact, the design was later made of greater weight than was required by computation.

Perfectly successful means are available, however, for water-proofing the structure so as to prevent uplift, using a membrane of lead, zinc, copper, or asphalt and cloth. Of necessity, any such material must be covered on the up-stream face, hence it is proposed to embed it about 3 ft. from the back, with

* "Report on New York's Water Supply," New York, 1900, p. 365.

† *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 154.

steps formed in the concrete where possible and inclined tie-rods to ensure that this covering will act as a part of the main body of the dam. Fig. 1 shows clearly how this may be done.

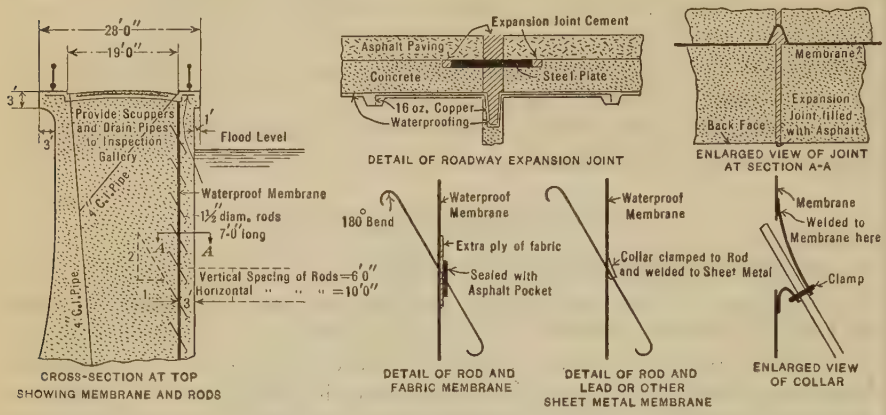

FIG. 1.

Of course, it is assumed that the foundation bed will be tight. If this proves not to be so, then of necessity drainage galleries could not be provided without causing serious leakage; but why build on such a foundation? If the foundation is not entirely tight, it is proposed to carry down the usual cut-off wall, with a water-proofing membrane within it.

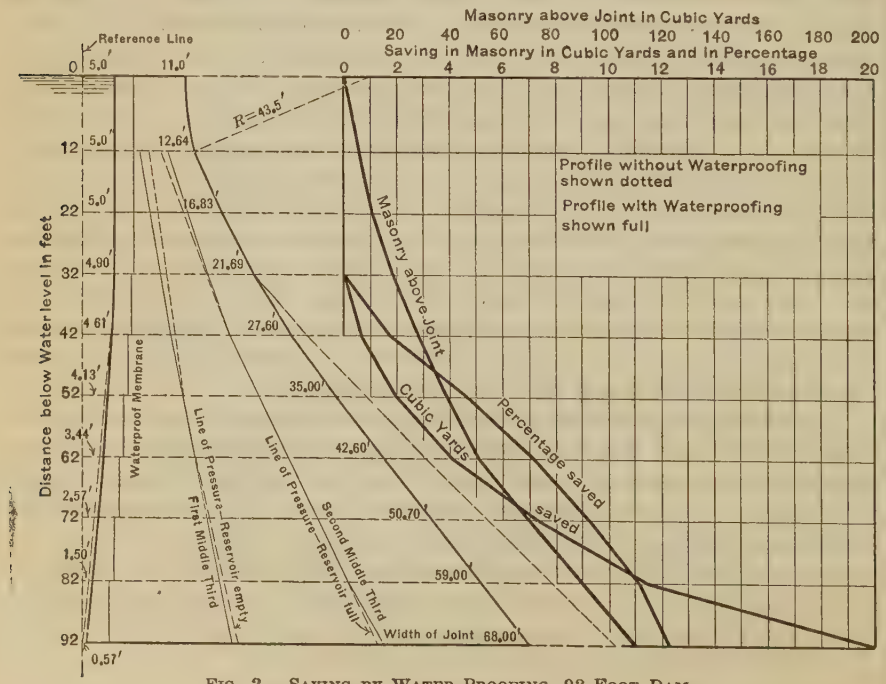
Masonry above Joint in Cubic Yards
0 20 40 60 80 100 120 140 160 180 200
Saving in Masonry in Cubic Yards and in Percentage
0 2 4 6 8 10 12 14 16 18 20
Reference Line
Distance below Water level in feet
R=43.5'
Profile without Waterproofing shown dotted
Profile with Waterproofing shown full
Masonry above Joint
Cubic Yards
Percentage saved
Waterproof Membrane
Line of Pressure First Middle Third Reservoir empty
Line of Pressure Reservoir full Second Middle Third
Width of Joint 68.00'
0.57'

FIG. 2.—SAVING BY WATER-PROOFING, 92-FOOT DAM.

When mechanical means are provided for meeting a condition instead of using increased strength or mass, it is good judgment to provide some method of checking from time to time their successful operation. Hence, if such a membrane is installed, drains and galleries should be provided. With such passageways leakage could not cause uplift, but clearly the small cost of water-proofing compared with the saving in masonry and the desirability of keeping water away from all structural concrete both justify the use of such a membrane.

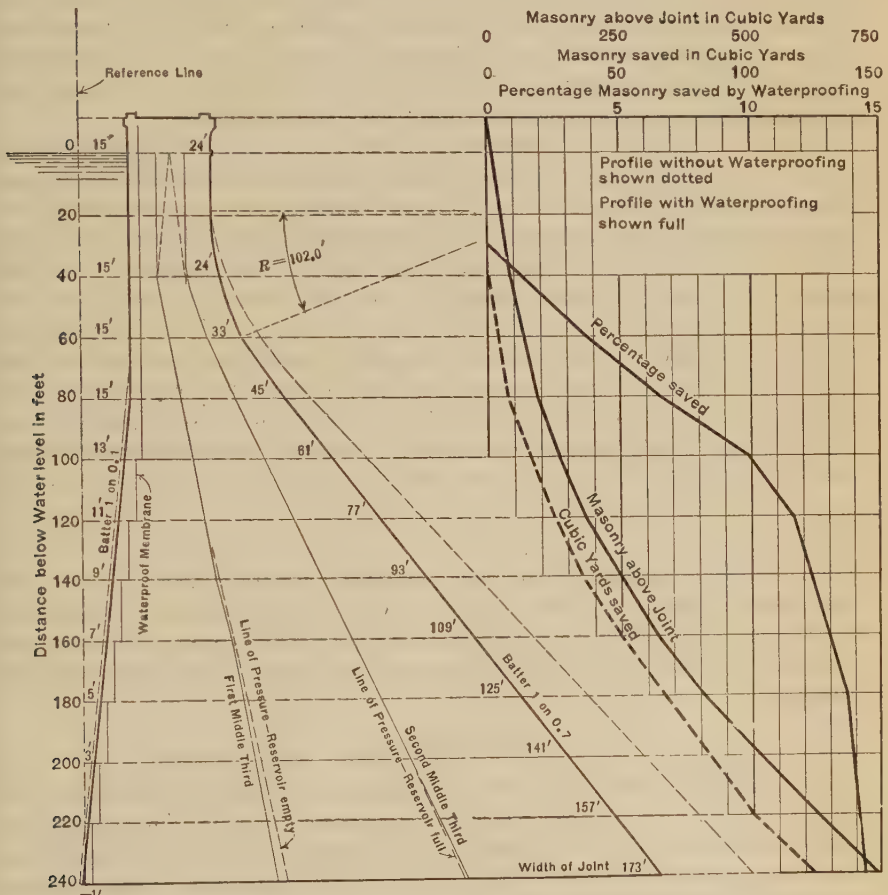
Masonry above Joint in Cubic Yards
0 250 500 750
Masonry saved in Cubic Yards
0 50 100 150
Percentage Masonry saved by Waterproofing
0 5 10 15
Reference Line
Profile without Waterproofing shown dotted
Profile with Waterproofing shown full
Percentage saved
Masonry above Joint
Cubic Yards saved
Distance below Water level in feet
0 20 40 60 80 100 120 140 160 180 200 220 240
15′ 15′ 15′ 15′ 13′ 11′ 9′ 7′ 5′ 3′ 1′
24′ 24′ 33′ 45′ 61′ 77′ 93′ 109′ 125′ 141′ 157′
R = 102.0′
Batter 1 on 0.1
Waterproof Membrane
Batter 1 on 0.7
Line of Pressure—Reservoir empty
First Middle Third
Line of Pressure—Reservoir full
Second Middle Third
Width of Joint 173′

FIG. 3.—SAVING BY WATER-PROOFING, 250-FOOT DAM.

For a low dam the economy of water-proofing will be smaller than for a high dam, because the cost of the membrane varies directly with the height, while the saving in masonry varies as the square of the height. Hence, while in a low dam the cost of the membrane might conceivably be as much as the saving in masonry, it would form only about 3% of the cost for a dam 100 ft.

high, as against a saving of masonry of about 12%, or a net saving of 9%; and for a 250-ft. dam the cost would be perhaps 1% against a saving of 14%, or a net saving of 13 per cent.

Application of This Idea to Two Specific Cases

Thus far the argument has been based on theoretical profiles, but it is now proposed to consider two specific designs, one for a low dam (about 100 ft. high, Fig. 2), the other for a high dam of about 250 ft. (Fig. 3), to determine how near to the theoretical saving it is possible to attain. The savings should approach 15.5% as a limit for increasing height, because at low heights the top width of dam and not the stability is the controlling factor. For each dam a design is made both with and without uplift, and in the latter case allowance is made for the concrete protection coat over the membrane. The limiting conditions of loading are for the reservoir both full and empty.

The top width is based on practical considerations, also the height from the reservoir level to the top of the dam. These dimensions were arbitrarily chosen. Computations then determine the depth at which for this top prismatic portion the pressure line, reservoir full, passes beyond the middle-third point. Below this the front face is battered. Similarly, below the depth at which the pressure line, reservoir empty, passes beyond the inner third point, the upstream face is battered. Ice pressure has not been considered for reasons previously discussed. Uplift has been assumed to vary from two-thirds of the hydrostatic head at the heel to zero at the toe, although if there were any considerable tail-race depth, the drainage galleries would have to be placed above water to permit inspection, and there might be pressure at the toe greater than zero. The weight of masonry was taken at 146 lb. per cu. ft., making $\Delta = \frac{7}{3}$.

Plotted beside each profile in Figs. 2 and 3 is a diagram showing quantities and savings in masonry. The respective designs are superposed to show the direct comparisons. Clearly the theoretical percentage is being approximated for the greater heights, the saving being 14% for a 250-ft. dam against 15.5% theoretical.

Conclusions

The maximum theoretical saving of masonry is 15.5%, based on a triangular design. The profiles shown contemplate no saving in the top, or parallel, portion because the width of this is fixed by independent conditions. Beginning about 30 ft. below reservoir level, the water-proof dam contains less masonry, and increasingly so, for greater depths. At the 92-ft. depth of the low dam about 12.3% has been saved; and at the 250-ft. section of the high dam, 14.3 per cent. A study of the curves will show that the amount of saving approaches the theoretical maximum as a limit, although probably the actual saving in a real structure would never exceed 15 per cent. At this 250-ft. level, however, 123 cu. yd. have been saved per linear foot of dam; in other words, for a dam 500 ft. long and 250 ft. high, with concrete at $10 per

yd., there would be saved 61 500 cu. yd., or $615 000. After deducting the extra cost of water-proofing, there would still remain a net saving of more than $500 000; hence it would seem that this method of eliminating upward pressure is well worth while in any large dam more than 50 ft. in height.

Acknowledgment

The burden of the preparation of the material was carried by Mr. Guy Lee Bryan, to whom it was suggested as topic for a graduation thesis leading up to the degree of Master in Civil Engineering in the Johns Hopkins School of Engineering. All computations and the preparation of the thesis were under the writer's direction.

DISCUSSION

KARL R. KENNISON,* M. AM. SOC. C. E. (by letter).—Although the author has called attention to the necessity of considering uplift in the bed-rock itself as well as in the masonry, this necessity is not sufficiently emphasized. The importance of this point should be stressed because a tendency has often been noted in such designs to assume that uplift could be entirely eliminated by proper preparation of the foundation or a proper contact between the masonry and bed-rock.

The author apparently assumes that the foundation will be tight; "else why build on such a foundation?" The usual problem, however, is to take the best foundation available, usually without much choice, and build the dam to suit the conditions.

In analyzing the stresses in a dam the writer has never emphasized the distinction between the masonry and the bed-rock, but has always seen to it that the lines of pressure could be safely carried well into the bed-rock. Having this in mind, the proper distribution of dead weight is required above any plane in the bed-rock, as well as above any similar plane through the masonry, and on that account it would be difficult to save as much yardage of masonry as the author indicates because the weight is needed on the bed-rock.

The writer does not recall any failure of a concrete gravity dam by overturning on a plane well up within the masonry itself. On the other hand, such dams overturn on their foundations or even on a plane below their foundations, the artificial concrete being more suitable, as far as preventing wholesale uplift and overturn is concerned, than the bed-rock itself, which is seldom (if ever) a dense homogeneous mass.

However, even if the cost of the up-stream water-proofing proposed cannot be saved in reduced yardage of masonry, it is, of course, very desirable to secure this water-tightness at the up-stream face, particularly to prevent slight seepages which can prove very unsightly without endangering stability and which in freezing weather can build up the full reservoir head and break off parts of the down-stream face. The cost of such water-proofing should be compared in each individual case with the cost of collecting seepage within the dam by various familiar methods.

P. A. BEATTY,† M. AM. SOC. C. E. (by letter).—In the Loch Raven Dam of the Baltimore, Md., water supply, on the Gunpowder River, designed, in part by, and constructed under the supervision of, the writer, a drainage system was installed to eliminate uplift.

This dam had a spillway height of 100 ft. above rock, and was constructed in 50-ft. sections, thus giving expansion joints at 50-ft. intervals along the axis of the dam. A carefully constructed facing of 1:2:4 concrete, erected in 8-ft. vertical lifts, extended from the bottom of the cut-off trench to the spillway crest, the sections being keyed at the expansion joints by two deep

* Designing Engr., Metropolitan Dist. Water Supply Comm., Boston, Mass.
† Cons. Engr., Camp Hill, Pa.

keyways with asphalt painted faces, and thoroughly bonded with the backing, which consisted of 1:3:6 concrete, containing about 25% of pudding stone in sizes up to 1 cu. yd., and also constructed in 8-ft. lifts. On the line of the expansion joints, at the junction of the facing and backing, vertical shafts, 30 in. square, extending from the first horizontal joint above the rock line to 8 ft. below the crest, were drained to the down-stream face of the dam.

From shaft to shaft was cast a groove. 14 in. wide and 7 in. deep, at each horizontal construction joint, and in this were laid porous concrete blocks, 3 ft. long and 14 in. square, perforated from end to end with a 6-in. circular opening, forming a free passageway between the shafts, the theory being that water seeping along the construction joints would find and follow this line of least resistance to the shafts, that coming through the expansion joints following directly to the shaft, thus making impossible the building up of pressure within the structure of the dam. Drainage was provided also at the rock line to intercept any flow passing beneath the cut-off wall.

The writer has no knowledge of the use of a water-proof metallic membrane in connection with any but rock-fill dams, but the saving in concrete pointed out by Mr. Pagon will undoubtedly direct further attention to this mode of construction. Practical consideration of the matter suggests the following questions to be studied:

What danger might result from the difference in the coefficient of expansion between the concrete and the metal used?

What would be the result of very small leaks in this membrane considering the numerous ties required to bond the two sections, that is, the backing and the protective coating, and the danger of perforation during construction?

In case the pool dropped, exposing a considerable area of the dam during very cold weather, what effects might be expected from successive freezings, recognizing the tendency of moisture to retire before cold, through a porous material, until an impervious surface is reached?

R. W. Gausmann,* M. Am. Soc. C. E. (by letter).—The author presents an interesting solution of the problem of gravity dam design. It is generally accepted that if water can be prevented from entering the up-stream face of a gravity dam no provision need be made for uplift pressure. The design proposed would undoubtedly accomplish this result providing a membrane could be made and kept absolutely water-tight at all times. It would be rather difficult to fabricate and test this membrane under field conditions and also protect it during the pouring of the concrete.

In addition, ample provision would have to be made not only for the expansion and contraction of the masonry but also for the gradual settlement that would occur during construction. Granted that all this might be accomplished, it is still a question whether the saving is as great as has been estimated by the author.

Generally speaking, masonry dams are not the same height throughout their length, in fact, the maximum height extends only for a short part

* Hydr. Engr., Ulen & Co., Athens, Greece.

of the length. This is particularly true of high masonry dams, therefore, the assumed saving should not be computed by multiplying the maximum section by the length as has been done by the author, but should be determined for each individual case.

A typical dam was computed using the author's graphs and costs and an actual profile across a valley. In this computation no allowance was made for foundations. The dam, constructed with a membrane, showed a saving of 6.0%, whereas the author's method shows an apparent saving of 12 per cent. A rough computation of the probable cost of a lead membrane (considered to be the most satisfactory) reduced the saving from 6.0% to a little more than 2 per cent.

It seems to the writer that the masonry outside the membrane should not be considered a part of the structure proper unless a much greater number of ties was used than is indicated on Fig. 3; it should rather be considered as a veneer or protection for the membrane. If this is the case then the dam must be widened by the thickness of this protective masonry and this added width will reduce the saving to almost nothing.

Should the membrane be punctured by some unforeseen contingency or if, due to faulty workmanship or poor material, it should not be absolutely water-tight, there is no provision in the design to care for the uplift which is bound to occur.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—Of the three features "not to be overlooked" it appears that uplift should be regarded as the least troublesome or uncertain. At Berea, Ohio, there is a deep sandstone quarry. In the fall massive blocks, 2 cu. yd. or more in size, may be taken from the bottom of the quarry, shipped hundreds of miles, and properly set as ashlar masonry. Then after the first freeze each block will be found neatly divided on a horizontal plane. This fact is mentioned to show that under pressure at great depths and in long periods of geologic time the sandstone becomes saturated with water much more thoroughly than it would ever be near the earth's surface. The quarrymen know that the output from the bottom of the pit should be "seasoned" before it is sold.

Now, imagine a dam of any reasonable cross-section carved out of like saturated sandstone, free from cracks or seams but subject to water pressure throughout. The conditions for promoting uplift would thus be ideal or perfect. Pass a horizontal plane through such a dam at any elevation and the uplift necessary for a change in position would be measured approximately by the weight of that part of the dam above the plane plus the cohesive strength of the sandstone. The resistance to uplift would be definite and well co-ordinated, like the weight and wind stresses in an anchorage. Substitute concrete, or cyclopean or other masonry, for the natural rock and although saturation might not be so perfect the tensile strength of the material as a mass would become an essential if not a controlling factor.

The point is that uplift is a variable. Bad construction, great uplift; good construction, small uplift! This applies to foundations and anchorage

* Civ. and Hydr. Engr., New York, N. Y.

as well. The Austin (Tex.) Dam gave way after a 5-in. rainfall and when the water was higher than previously known. At that time the vertical difference in elevation between the water below and above the dam was doubtless less than it had been before. That vertical distance was the measure of the uplift due to pressure and was also less than before. The high water lessened the weight of the dam and its force moved a part of the structure down stream.

Back-water at a flood stage of any river, even the Niagara, is noticeable. In a Kansas river a dam 12 ft. high could not be located by the appearance of the surface of the water directly above it during a flood stage. The difference between low and high water was about 32 ft. Under those conditions there was no uplift.

H. de B. Parsons,* M. Am. Soc. C. E.—All dams are intended to be water-tight, that is, tight against leakage, but not necessarily tight against percolation of water. Earthen dams and rock-filled dams are not expected to be tight; but as they are not "masonry" dams, they are without the scope of this discussion.

In broad conception, there are three elements entering into a design for a dam, that is, a water-tight structure, a mass to give strength, and a shape to effect economy in construction. Masonry dams are built of stone, or concrete, or both in combination. They are intended to be water-tight, and an effort always is made to make them water-proof. This effort is carried as far as considerations of economy will permit. Large dams of gravity cross-section, as illustrated in the paper, almost always are made of stone or of stone facings with cyclopean concrete backing. The Spier Falls Dam across the Hudson River, for which the speaker made the designs, was of the latter character. Artificial stone is used at times in place of natural cut stone.

To build an effective water-proof membrane in a stone dam would be extremely difficult. The stones overlap and break joints, the shapes of the sides and backs of the stones are irregular, and they are placed so as to interlock one with another. To construct in the masonry a water-proof membrane, such as is mentioned in the paper, would seem to be a difficult operation.

A water-proofing membrane of lead, copper, zinc, or cloth, as described by the author, therefore, seems to be limited to mass concrete structures and not to those of stone or concrete blocks. The title of the paper would appear to be misleading, if the speaker's conception of the suggested membrane be correct. The section of an all-concrete dam is usually different from the gravity sections shown in the paper. Arch dams, multiple-arch dams, buttress dams with aprons, and similar designs achieve savings in concrete that are as great as or greater than the percentages mentioned by the author. Economy in the quantity of concrete used, and in the cost of its placement, depends on the shape and form of the section selected. The cost of wooden forms must not be overlooked, and the additional cost of

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forms for the insertion of a water-proofing membrane, as illustrated, might be a serious factor.

Concrete will not hold fast to and bond with smooth flat metallic surfaces; and, in the speaker's opinion, it is wise that the author has suggested the use of diagonal tie-rods. These rods, passing through the water-proofing membrane, will tie together the concrete masses separated by the membrane.

The suggested membrane materials have different linear rates of expansion. Taking concrete expansion as 100, copper has a relative expansion of about 121, lead about 188, and zinc about 203. In other words, the expansion under temperature changes of a copper membrane would be more than 20% greater than concrete, and that of a lead or zinc membrane even more. The difference in expansion of concrete and a metal membrane will result in rupture or separation of the concrete mass from the metal surface. The ranges of temperature to which the water-proofing membrane might be exposed in a climate like that around New York, is roughly 30° Fahr., say, 32°, for the minimum temperature of the water, and 62° for the maximum. For that part of the membrane above the water-line, the range of temperature would be greater.

Since the 3-ft. concrete section back of the water-proofing membrane might separate from and act independently of the main concrete mass, it is questionable whether engineers would include this section in their calculations for strength of such high dams as those illustrated in the paper.

Practically all stone dams (whether natural or artificial) are more or less porous. Luckily the tendency is for silt to find its way into the pores of the stone, and gradually close them. In a natural stone or artificial stone dam, the leakage, if any, is through the joints. After a dam is built, joints between stones and, in a concrete dam, construction and expansion joints need careful watching. The same care would be necessary to watch the membrane, which must have many joints between the copper, zinc, or lead sheets. These metal joints would exceed in number the joints in a mass concrete dam, where they are limited to expansion and construction joints.

Although ordinary concrete is porous it can be made almost water-tight by proper proportioning or working of the ingredients. Therefore the thought arises, would it not be cheaper, that is, more economical, to regulate closely the aggregate, cement, lime, and water of the mix, than to construct a water-proofing membrane as proposed?

There are compounds which can be applied to the surfaces of concrete that will almost render it water-tight or water-proof. In the speaker's experience the greatest benefit from these applications results from the careful inspection of the surface of the concrete, while smoothing it, brushing it, and filling up the holes left by the forms.

In a concrete dam it is difficult to keep water out of construction joints and shrinkage cracks. When water reaches the reinforcement, the metal rusts and, in rusting, produces an expansive force that breaks the concrete. This trouble is usually noticed between "wind and water", that is, where the reinforcement is alternately wet and dry.

Ice is another serious cause of trouble. The impact of ice floes breaks the hard surface of the concrete, and opens channels for water to enter into the mass. If it does, the water then will reach the reinforcement.

The author assumes the foundation to be tight, and he states that "if not, why build on such a foundation?" Unfortunately, the engineer often has to build a structure on a pervious foundation. If the foundation is pervious, or if water enters under a dam through fissures in the rock, either under or at the sides, there will be uplift at the base which a water-proofing membrane will not prevent. With an uplift pressure it would seem wiser to expend money in trying to control the uplift, rather than to construct a water-proofing membrane, such as described, which would be difficult to make tight at the joint with the foundation material.

What the speaker has said should be taken not so much in criticism of the paper as a statement of the difficulties of construction and maintenance as they appear from the standpoint of design and construction in the field. It is hoped that mentioning facts which may seem in opposition will encourage debate and discussion. To design and construct a really water-proof dam would undoubtedly be a great achievement.

Alfred D. Flinn,* M. Am. Soc. C. E.—Such experience as the speaker has had in designing and building dams leads him to doubt seriously the practicability of successfully incorporating in a dam a water-tight diaphragm or membrane, as proposed by the author. The placing and safeguarding of the membrane and anchors (Fig. 1) are not compatible with economical methods of building massive bodies of concrete, cyclopean, or other kinds of masonry used for large dams.

Other considerations than prevention of uplift led to the provision of drainage pipes, wells, and galleries in some well-known dams. One purpose was to reduce or prevent the slow local deterioration of the masonry by the leaching action of water wherever it could seep through. Another purpose was to prevent or reduce disfiguration of the down-stream face of the dam by seepage. Still another purpose was to prevent suspicions as to security sometimes aroused in lay minds by the appearance of seepage on the face of the dam. Some reduction of uplift may be gained also, but drainage passages cannot be depended on permanently because the leaching and depositing of salts from the masonry tend to seal the designed and accidental passages.

J. B. W. Gardiner,† Assoc. M. Am. Soc. C. E.—The speaker would like to discuss one point that has already been mentioned several times, that is, the method of fastening the protecting face against both the water-proofing and the main body of the dam. The author's design (Fig. 1) shows the water-proofing to have been punctured in many places by the rods used in tying the two parts of the dam together. There is a great element of danger, particularly where heavy hydrostatic heads have to be combatted, in puncturing a water-proofing blanket and, as already pointed out, it has not proved

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† Pres., J. B. W. Gardiner, Inc., New York, N. Y.

satisfactory in practice. It is the speaker's opinion that any water-proofing plan should be so designed as to avoid such puncturing.

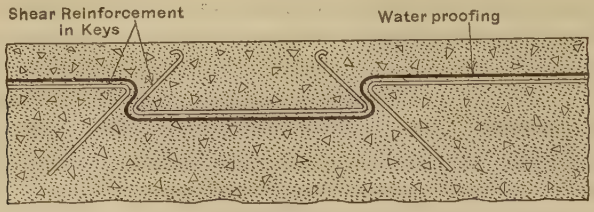

FIG. 4.

In this case a design that suggests itself as accomplishing the desired end is shown in Fig. 4. Keys, as shown, would be cast in the face of the main body of the dam, running vertically to facilitate the placing of the concrete face. The strength of shoulder necessary to take care both of anchor ice and of the tendency to overturn, could be calculated and shear reinforcement placed as shown. The water-proofing would be carried down into the key and the entire mat over the face of the dam could be made a continuous unbroken sheet, without breaks or punctures. The speaker does not believe that the cost would exceed that of the author's design, while the efficiency would be vastly increased.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—It is difficult to conceive of any membrane water-proofing under a dam that might not do more harm than good.

One reason for this is that most membrane water-proof materials are liable to be destroyed in time—leaving a space for the water to spread—thereby greatly increasing the uplift. Copper itself will deteriorate under certain conditions, and even if it did not, it could not be hammered down on the bed-rock so that spaces would not be left beneath, which would be filled with water.

In New York, bed-rock is practically impervious, but fissures are frequent with a full head of water which, if allowed to spread under the base, would give an uplift proportional to that spread.

There was a case in the West, where, if the entire dam had been water-proofed from the outside—top, bottom, and sides—it might have been saved, regardless of uplift. That was a steel dam, the first cost of which was 10% less than a masonry gravity dam. A few months after construction, the dam collapsed, and it was then found that a 10-lb. hammer had been almost completely destroyed by the acids in the water.

Theoretically, the worst conditions can be calculated, provided a few important factors are not overlooked; but to try to guess at the actual uplift and ice pressure requires more than a calculating machine. For example, all the Hudson River Tunnels are lighter than the water displaced, and it cannot be proved that they will stay "put", nor the reverse.

* Cons. Engr., New York, N. Y.

Again, a pneumatic caisson was founded on rock and had its air chamber filled with concrete when it broke loose, rose, and moved, due probably to the fact that the concrete had been stopped 4 ft. below the intended elevation before sealing the air chamber. Even the adhesion of the concrete to the bed-rock was not sufficient to overcome the uplift, although another foot of concrete would have been sufficient. A pneumatic caisson in New York had 4 ft. of concrete placed in the air chamber and the weights taken off with an utter disregard for uplift, and up it came.

The speaker hopes to build a dam where the ice conditions are probably the worst in the country, and to make it safe by having a slope on the up-stream side so flat that the ice cannot create any pressure against the dam, and with a similar flat slope on the down-stream side so that the ice will slide down into deep water, instead of, as usually happens, dropping from the top of the dam and undermining the foundation.

V. Bernard Siems,* M. Am. Soc. C. E. (by letter).—In this paper, Mr. Pagon has proposed an interesting subject. His treatment is careful, complete, and novel. However, certain questions immediately arise as to the desirability of the plan which he has suggested. Whether to sacrifice mass in dam structures for the purpose of reducing cost by substituting an absolutely impervious up-stream facing consisting of a membranous coating is, to the writer, an open question.

The author's design inserts in the dam structure an artificial cleavage plane, an opportunity for mass faulting, for it is questionable whether or not the face mass could be anchored to the body of the masonry to prevent a vertical plane of excessive weakness in shear. With the separation of the face mass from the masonry body by a frictionless membrane, the two masses, notwithstanding the anchors, could not be considered as acting monolithically. The writer also doubts the permanent tightness of the membrane, especially at the point where the bars pass through it, or at any puncture, for such points would be difficult to seal completely. He also believes the coating would be difficult to apply.

Certainly efforts should be made to obtain impervious concrete with a hard outer surface resistant to frost action. Concrete deteriorates when water penetrating beyond its surface dissolves the calcium compounds, and permits frost and ice action to disintegrate the concrete mass mechanically. The method of building the facing is not entirely clear. If built after the main body of the dam, no doubt the thin facing would be more expensive than its volume would indicate. A well-designed and constructed concrete dam should not have any large percentage of its interior subjected to uplift by water. No doubt a dam the face of which has been constructed of a rich mix, with its inner mass of an 1:3:6 mix, would absorb in the interior mass any small leakages that pass through the exterior base without exerting uplift. If, as Mr. Pagon suggests, inspection and drainage galleries are built, the upward pressure will be relieved.

* Vice-Pres. and Gen. Mgr., North Am. Water-Works Corporation, New York, N. Y.

The chief trouble from uplift is in or under the foundation. The greatest damage from uplift would occur in case there were subterranean fissures beneath the dam into which water might find its way. Large dams should always be provided with porous drain-tile discharging into the gallery for the purpose of carrying off any leakage that might pass the outer and more impervious concrete.

Where large dams are built in thickly populated territory, an ample factor of safety should always be provided. The writer prefers to rely on mass rather than on a membranous water-proofing near the face of the dam, especially as the danger point may be beneath the dam, as is true in some parts of the world—India and Egypt, for instance—instead of in the interior mass.

He would rely, therefore, on mass and a good quality of concrete that can be secured by restricting to proper limits the quantity of water used, by the prevention of loss of cement, by the insurance of a proper mix, by proper tamping and keying, and by protection from injury by jar or frost.

B. F. Jakobsen,* M. Am. Soc. C. E. (by letter).—The Los Angeles County Flood Control District has under consideration a gravity dam nearly 500 ft. high on the San Gabriel River, for which $25 000 000 was voted in 1924. The present design, known as Design R, was prepared by the Designing Engineer for the Flood Control District, Mr. S. M. Fisher and has received the approval of two consulting engineers, John D. Galloway and D. C. Henny, Members, Am. Soc. C. E. This design provides for 50% uplift at the heel and zero uplift at the toe.

A considerable saving, approaching 15%, or $3 750 000, can be effected, judging from the author's Fig. 3 and, therefore, the merit of the proposed remedy seems well worth investigation.

The author states: "Of course, it is assumed that the foundation bed will be tight." Unfortunately, however, if uplift is at all likely to be present, the writer's experience is that the joint between the rock and the concrete is the most dangerous point. If the proposed water-tight membrane costs as much as designing for uplift, it would most likely not be proposed; its only advantage is its cheapness.

The author states: "Since, therefore, no important dam would be built nowadays without considering uplift, the whole question of water-proofing appears to be important." The writer wishes to call attention to two major dams, the Don Pedro Dam and the Exchequer Dam, both located in the San Joaquin Valley, California, which have drainage galleries and cut-off walls with drains behind them, but otherwise have no provision for uplift included in the design. It was evidently assumed that the provisions made were so successful as to make it safe to assume that there could be no uplift. A. J. Wiley, M. Am. Soc. C. E., was the Consulting Engineer for both projects. Whether uplift could ever be present may be a mooted question on which competent engineers may disagree, but to assume no uplift is always less safe than to design for some uplift.

* Cons. Engr., Los Angeles, Calif.

It may be well to bear in mind that there are various degrees of probability, as, for example, it is highly probable that 1 cu. yd. of concrete will always, weigh, say, 2 tons; it is less certain that it will also have the estimated strength, and still less certain that tension can be transmitted across contraction joints.

There is some danger that the membrane may be damaged during construction and that water may accumulate at the up-stream face of the membrane and possibly detach the concrete from it. In computing the stresses in a gravity dam provided with such a skin, the writer would insist that the section be reckoned from the skin down stream and that the 3 ft. of concrete lying up stream from the membrane be excluded.

When considering what can be done to lessen or eliminate uplift, it should always be had in mind that to design a dam of a sufficiently heavy section is the best and most positive remedy. To provide drains may be considered in many instances at least as a partial remedy, but there is always the danger that drains may clog, while there is not much danger that the material will lose in weight. To assume outright that uplift cannot occur, or to assume that drain pipes will always work, may sometimes amount to the same thing, and the engineer should guard against just such a contingency.

The author obtains his saving by comparing a design good for a two-thirds full hydro-static pressure at the heel, but without a skin, with one provided with a water-tight skin, but otherwise having no allowance for uplift. The writer cannot agree that these two designs are even approximately equally meritorious and, moreover, he would always design a high gravity dam so that there could be no reasonable question about whether or not tension exists in the up-stream face when the reservoir is as completely filled as it is ever likely to be.

The middle-third theory assumes that horizontal sections remain plane during deformation, which is certainly not an exact assumption, but on the other hand is not likely to be far wrong. The general bending theory assumes that normal planes remain plane and this assumption excludes the one that horizontal sections remain plane, since these are not normal but oblique to the axis of the structure.* Moreover, the assumption that normal sections remain plane, has been verified, as far as the writer knows, only for small sections, say, those of a few inches in thickness, and is true only when no shear, or a uniform shear, exists. The assumption may not be exact and some caution is justified when applying it to a section several hundred feet thick and with unevenly distributed shear.

As far as the writer is aware all stress determinations made to date assume either that horizontal sections remain plane,† or that the normal

* "Stresses in Multiple-Arch Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 289 and 292, and also the discussion by William Cain, M. Am. Soc. C. E., p. 316.

† See "Stresses in Masonry Dams," by Ottley, Brightmore, Wilson, Gore, and Hill, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–08, Part II. Also, "Stresses in Masonry Dams," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

planes remain plane.* They all disregard the warping of the section due to shear stress and the influence that this may have on the distribution of the bending stresses.† For high dams designed without uplift, this influence may not be negligible. Also, experiments made to verify the fundamental assumption that normal planes remain plane, have been made on beams of uniform section; but a gravity dam is not such a beam and moreover it is fixed along an oblique plane, which is itself deformed by the stresses imposed on it.‡ Also, swelling due to water soaking, shrinkage, temperature variation, irregularity of foundation, etc., will have some influence on the stress distribution.

These various defects in the ordinary bending theory are pointed out for the reason that the assumption of considerable uplift enables the designer to neglect these errors and to treat the ordinary bending formulas as approximate, which they are. The stress in the heel of a dam when the reservoir is full is due to the difference between the stress when the reservoir is empty and the bending moment produced by the water when the reservoir is full. For a dam 500 ft. high, with a vertical up-stream face and a triangular section, designed with no uplift, and with concrete weighing 144 lb. per cu. ft., the stress in the up-stream face when the reservoir is empty is 500 lb. per sq. in., and the tension produced by the bending moment due to the water load is also 500 lb. per sq. in., so that the resulting stress in the up-stream face with reservoir full is zero. It is evident that if the bending stress is under-estimated by 5%, due to a stress distribution different from that assumed, a tension of 25 lb. per sq. in. exists in the up-stream face of the dam. This would not in the writer's opinion be a safe structure even if uplift could not occur. If uplift had been included in the design, however, the error in the ordinary bending theory would normally only reduce the compression in the up-stream face and would be no very serious matter. This same error would also effect the maximum stress in the down-stream face, but that would be of but slight importance, since it may be assumed that the error is not large.

Until the actual stresses in high gravity dams have been determined from experiments and theory, engineers will do well to bear in mind the possible influence of the inaccuracies in the fundamental assumptions. The proposed water-tight membrane may keep water out of the structure—except near the foundation—but it does not provide any protection against the factor of ignorance arising from approximations in fundamental assumptions.

Eugene E. Halmos,§ M. Am. Soc. C. E. (by letter).—The history of the development of the rational design of gravity dams, no matter how brief, should not omit mention of the work done by Maurice Levy and Karl Pearson. No

* "Formes et Dimensions des Grands Barrages en Maçonnerie," by M. Résal, *Annales des Ponts et Chaussées*, II, 1919.

† See discussion by J. P. J. Williams, M. Am. Soc. C E., on a paper by H. S. Prichard, M. Am. Soc. C. E., entitled "Faults in the Theory of Flexure," *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 932.

‡ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Fig. 45, p. 561.

§ Chf. Engr., Parklap Constr. Corporation, New York, N. Y.

gravity dam of importance, built in the last twenty or twenty-five years, was constructed without due consideration having been given in the design to the principles enunciated by these men. Their names are as intimately connected with the status of the present knowledge on stresses in gravity dams as those of Lord Kelvin, Crookes, and Steinmetz with the development of modern electrical engineering.

Modern dam design, based on the researches of Levy and Pearson and on model experiments demonstrating the truth of their conclusions, recognizes the fact that Rankine's criterion, namely, that the resultant of the forces must fall within the middle third of the section, is not sufficient to prevent the occurrence of tension in the mass of the dam. There may be tension in the masonry on planes other than horizontal even if all the stresses on the horizontal planes are compressive. To prevent tension it is necessary to design the dam so that there will be a substantial compression at the up-stream edge of any horizontal plane section. This compression must not be less than the intensity of the water pressure at the depth of the section investigated. This criterion, in the case of high dams, results in a materially heavier cross-section than that complying with the middle-third rule only, and such an enlarged section is necessary without regard as to whether or not the back of the dam is treated with water-proofing paint or is supplied with a water-tight membrane. On the other hand, compliance with this criterion will be entirely sufficient to make the dam safe as long as the uplift at the bottom is given proper consideration in the design.

The writer believes that the water-proofing of gravity dams for the purpose advanced by the author is entirely useless and may be a source of danger if the section is diminished from that necessary to prevent tension from occurring in the dam. In no case would he approve of a design which would consider the concrete covering of the proposed water-proof membrane to act as part of the dam.

The writer cannot help but criticize a tendency, fortunately not too frequent among engineers, to try to save money by the use of makeskifts at the risk of endangering the safety of important structures on which the life and prosperity of many human beings depend.

Edward Wegmann,* M. Am. Soc. C. E.—The author proposes to make uplift in a masonry dam impossible, by placing in the body of the dam, near its up-stream face, a water-tight membrane of metal, asphalt, or some other suitable material, and by carrying this membrane down to an impervious foundation.

In addition, he recommends the somewhat customary system of underdrainage, to ensure the removal of any seepage from the foundation bed, which he assumes to be practically water-tight. If this should not be the case, the drainage system might cause serious leakage, but the author asks: "Why build on such a foundation?" If the foundation is not entirely water-tight, he recommends the usual cut-off wall, with a water-proofing membrane within it. To what depth the cut-off wall should be carried down, he does not state,

* Cons. Engr., New York, N. Y.

but it is evident that the value of his system, if found to be perfectly practical, would only apply to cases where a water-tight foundation could be had at a reasonable depth.

The water-tight membrane would, of course, have to be tied to the remaining part of the dam by rods, etc., so as to enable the whole mass of masonry to act as a unit, and to prevent all possible leakage past the membrane. If this can be done successfully, the additional strength often given to masonry dams to prevent uplift can be greatly reduced and, possibly, omitted entirely.

There are a number of practical difficulties in placing a water-tight membrane in the body of a dam, as fully discussed by others. After all, the only way of determining positively the value of the water-proofing system proposed by the author is by actually building a dam with a water-proof membrane. It is to be hoped that in the near future he may be able to build such a dam.

The paper re-opens a question much discussed by engineers about twenty years ago, and about which there is still quite a difference of opinion, namely, does uplift exist in an ordinary masonry dam, built on a good, fairly water-tight foundation? and if it does exist, how much additional strength should be given a dam, to enable it to resist such upward pressure?

Prior to 1895, there was not a case on record in which the failure of a masonry dam had been attributed partly, if not entirely, to upward pressure. Distinguished foreign and American engineers had written memoirs and books, discussing the proper manner of designing masonry dams, but none of them, as far as the speaker knows, had mentioned the fact that with a bad, pervious foundation, uplift under the base of a dam might exist. Moreover, these engineers had designed and actually built about thirty masonry dams, in different parts of the world, without paying any attention to a possible uplift under the bases or to ice pressures. These dams range in height from 50 to 164 ft., and some of them are now more than 60 years old. None of them has failed.

On April 27, 1895, the Bouzey Dam, built in 1878-81 near Epinal, France, failed, causing a great loss of life and property. This dam was founded on red sandstone, which was much fissured and quite pervious. Much trouble had been experienced in the foundation trench from springs, and to prevent leakage under the dam a cut-off wall, 2 m. thick, was built at the up-stream face. The failure of this dam has been attributed to a greater tension at the up-stream face than the masonry could withstand, this pressure being, probably, increased by uplift.

In December of the same year, the late J. D. Van Buren, M. Am. Soc. C. E., read before the Society a paper,* in which he recommended that all high masonry dams should be designed for a full uplift—due to the whole head in the reservoir—acting on the base and on each assumed joint above it, and, also, in northern latitudes, for an ice pressure of 40 000 lb. per lin. ft., acting at the crest of the dam. Mr. Van Buren submitted a plan for a dam, designed according to his principles; for a height of 250 ft. it had a base of 352 ft.

* "Notes on High Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. XXXIV (1895), p. 493.

Considering the fact that of all the masonry dams built, including a number in which a large amount of uplift and ice pressure were considered in the design, none has a height equal to its base, the folly of taking such extreme assumptions is evident.

There was much discussion among engineers about Mr. Van Buren's paper. It was pointed out that according to his assumption, the dam would be floating, and that while with a badly fissured foundation, there might be considerable uplift at seams and fissures, it was not logical to assume this uplift as extending over the whole base. It was probably limited to a certain percentage of the foundation, and certainly did not extend under the whole base with the full head that was in the reservoir.

The construction of the New Croton Dam, the height of which was to exceed that of all existing dams by more than 100 ft., had been begun in September, 1892, and it was an important question for the late Alphonse Fteley, Past-President, Am. Soc. C. E., the Chief Engineer of the Aqueduct Commission of New York, under whose direction the dam was to be built, to determine whether, in view of Mr. Van Buren's paper and the discussion thereon, he should modify the profile of the New Croton Dam to include an allowance for uplift and for possible ice pressure.

The speaker was Mr. Fteley's Assistant in the design of the New Croton Dam, and is, therefore, familiar with the reasons which influenced his decision.

The four dams given in Table 1 had, up to that time, been constructed on the Croton water-shed, the last three being designed and built under the supervision of Mr. Fteley.

TABLE 1.—Dams on Croton Water-Shed Prior to 1892.

Name.	Built.	Maximum height above lowest point of foundation, in feet.
Boyd's Corners	1866–72	70
Sodom	1888–93	98
Titicus	1890–95	135
Carmel	1891–95	65

All these dams were founded on rock similar to that which was known to exist under the New Croton Dam. Mr. Fteley was of the opinion that any uplift which might exist under any of these dams, and which might occur under the New Croton Dam, would be limited to a small percentage of the area of the base. As regards ice pressure, more than a foot of ice had formed in the reservoirs of the dams given in Table 1, without causing any trouble. Mr. Fteley thought that any sheet of ice that might form in the New Croton Reservoir would buckle before it could exert a great pressure on the dam.

For these reasons, he decided to build the New Croton Dam without any allowance for uplift or ice pressure. This dam, which has a maximum height

of 297 ft., was completed in 1907, and to date has not shown the slightest sign of any weakness.

The next case of the failure of a large masonry dam due to poor foundation, undermining, and uplift occurred at Austin, Tex., on April 7, 1900, when the dam built across the Colorado River, about two miles above the city, was ruptured. This dam had been built in 1891-92. The water-shed above the dam contains about 50 000 sq. miles, consisting largely of mountainous country. For two or three days, a very heavy rainfall occurred in this water-shed, causing the river to rise until it flowed with a depth of more than 11 ft. over the dam. The dam broke, the current found its way through the gap and shoved two sections of the dam, each about 250 ft. long, bodily about 60 ft. down stream, without the slightest overturning, leaving them almost parallel with their original positions. During the night these sections were destroyed by the stream.

In this case, the dam had been built on limestone, which was in places so soft that it could be removed with picks and shovels. Moreover, it had been built directly over a geological fault, 75 ft. wide, full of adobe, with red streaks of clay, extending to an indefinite depth. The current passing over the dam had undermined the front toe. Under these circumstances there can be no question but that a large uplift was exerted under the dam.

In 1900-06 the late Frederic P. Stearns, Past-President, Am. Soc. C. E., and his assistants, designed and built the Wachusett Dam across the south branch of the Nashua River, to form a large storage reservoir for the Metropolitan District of Boston, Mass. This dam has a maximum height of 228 ft. above the lowest point in the foundation. Its cross-sectional area for the same depth is considerably greater than that of the New Croton Dam.

Discussing with Mr. Stearns why he made so great a difference, the speaker was told that the City of Clinton, Mass., was about ½ mile below the dam, that great apprehension of a possible failure was felt by the residents; and Mr. Stearns added: "If I had not made the dam unusually thick, I would never have been able to get my plans accepted."

The basis on which this increased thickness was computed was by estimating an uplift under the whole base of the dam, beginning with two-thirds of the full head in the reservoir at the heel and diminishing uniformly to zero at the toe; and also by assuming an ice pressure of 47 000 lb. per lin. ft., acting at the crest of the dam. Under the circumstances, Mr. Stearns, no doubt, was justified in assuming a very large factor of safety for the Wachusett Dam.

Soon after the completion of this important dam, Mr. Stearns was appointed as one of the Consulting Engineers of the Board of Water Supply of the City of New York, under the direction of which the Ashokan, Kensico, and Gilboa Dams were built. In designing these dams the same allowance was made for uplift as in the Wachusett Dam, and a large ice pressure was assumed. As the City of Kingston was about 20 miles below the Ashokan Dam, and the City of White Plains only 3 miles below the Kensico Dam, and as practically unlimited financial means were available, all will agree that

the engineers were justified in adopting for these dams unusually strong profiles.

Two more dams have been built in the Croton water-shed in recent years, namely,

	Built in.	Maximum height, in feet.
Cross River Dam.....	1904-08	153
Croton Falls Dam.....	1906-11	167

Both these dams were designed according to the principles adopted for the Wachusett Dam, the ice pressure being assumed at 24 000 lb. per lin. ft. for the Cross River Dam and at 30 000 lb. per lin. ft. for the Croton Falls Dam. Both these structures are located on affluents of the Croton Dam, about 15 miles up stream from the New Croton Dam, where there are few buildings and only a sparse population.

There exists, therefore, in the Croton water-shed this rather illogical condition: Five dams, including the New Croton Dam, the most important of all these structures, designed without taking uplift and ice pressure into account; and two dams designed by taking uplift and ice pressure into consideration, the latter having much larger profiles than the former, and, of course, involving a corresponding increase in cost. If the engineer had not to consider cost, his problems would be comparatively simple. By making the base of a dam equal to its height, he would be amply safe in all cases, even if uplift and ice pressure were to be considered. True skill in engineering consists, however, in obtaining the required safety with a minimum of expense.

There is a third case on record of a dam being ruptured by uplift. On September 30, 1911, the structure at Austin, Pa., failed, causing the loss of seventy-five lives and of much property. In this case the dam was built on a pervious foundation of shale, the excavation being only about 6 ft. deep, and no cut-off wall being provided. The first time the reservoir was allowed to be filled in part the dam was curved about 1 ft. down stream by the water pressure.

The engineer in charge of the work sought the advice of a consulting engineer, and was told that the only way to make the dam safe was either by building a good cut-off wall to an impervious stratum, or by constructing on the down-stream side of the dam a rock-fill dam, strong enough to keep the masonry dam in place. The engineer recommended that both these expedients be adopted and estimated that the cost might be $30 000, and possibly much more. He was promptly discharged and the President of the wood pulp and paper company for which the dam had been been built, who was what is often called "a practical man", took charge of the work. The reservoir was gradually filled. When the water reached the level of its spillway, the dam failed by sliding on its foundation, exactly as had been predicted by the consulting engineer. This failure was undoubtedly due to uplift.

To sum up the case as it stands at the present time:

More than forty masonry dams, in excess of 100 ft. in height, have been built in various parts of the world, and are still standing, without taking uplift and ice pressure into account.

Seven masonry dams, including the Hetch Hetchy Dam, have been built with large profiles, taking uplift and ice pressure into consideration.

Three masonry dams, all less than 70 ft. in height, have failed. All were built on poor, pervious foundations, and there can be no doubt that these failures were largely, if not entirely, due to uplift.

In conclusion, the speaker would state that whether or not uplift should be considered, is a question for the exercise of good judgment on the part of the engineer. It depends mainly on the character of the foundation. If it were as bad as that of the dam at Austin, Pa., the engineer might be justified in assuming a full upward pressure. Where large populations live below a dam and where great loss of life would occur in case of failure, the engineer could be justified in providing a much larger factor of safety than would be required in ordinary cases.

As far as ice pressure is concerned, the speaker knows of only one case in which this force was the primary cause of the failure of a dam. At Minneapolis, Minn., in 1893-94, a mill company had built a dam of ashlar masonry, to form a pond for supplying power. The dam was 535 ft. long, 18 ft. high, 12 ft. wide at the base, and 5.25 ft. wide at the coping. A retaining wall, parallel with, and 350 ft. distant from, the dam formed the opposite side of the pond. In winter, ice 4 ft. thick covered the pond. During the week the pond was usually drawn down 2 ft., or more, to meet the demand for power, but on Sundays the pond was allowed to rise to its normal level. This caused the sheet of ice to exert a toggle-joint pressure on the dam.

The dam stood successfully until the spring of 1899. In February of that year, it was noticed that the dam was slowly revolving around its down-stream toe, the top of the dam having moved 10 to 12 in. from its original position. There was no sliding.

The ice was cut out back of the dam, which returned slowly almost half way to its original position. On April 30, 1899, a section of the dam, about 170 ft. long, slid out. This failure is supposed to have been caused by water getting under the dam through some cracks which were opened when it was slightly revolved the preceding February. In this case there were peculiar circumstances: A sheet of ice, 4 ft. thick and 350 ft. long, abutting at the end against the dam, and at the other end against a retaining wall.

About 1916, the speaker was engaged by the Quebec Streams Commission to design a dam across the St. Maurice River, in a locality where ice, 6 ft. thick, formed every winter. The dam which was to have a height of 80 ft. was founded on Laurentian gneiss, which was as solid as granite, and entirely free from seams and fissures. No allowance was made in this case for uplift, but an ice pressure of 50 000 lb. per lin. ft. was assumed to act at the crest. The dam was built successfully and is still in use.

MALCOLM ELLIOTT,* M. AM. SOC. C. E. (by letter).—Water-proofing the up-stream face of a dam as proposed by the author would in many cases lead to construction difficulties which might offset the advantages. Simplicity should be sought in the design so that the materials can be placed at the least possible expense. Especially is this true in the case of work that is frequently menaced by bad weather and floods. The proposed membranes might cause serious delays to the construction program, increasing the hazards due to floods and weather so as to offset any saving in material that might result from the water-proofing.

It is believed, however, that when first-class construction is sought, water-tightness should be carefully considered and obtained by any reasonable details of the design that will not unduly delay or increase the cost of the work. So far as stability is concerned, many cases may occur in which the most economical method of counteracting upward pressure would be to increase the weight of the dam rather than to attempt to exclude the water. In any case, there is an element of danger in assuming that the water-proofing elements will be entirely effective. While membranes as proposed would probably cut off a large part of the leakage, the assumption that all water, and thus all upward pressure, would be eliminated, is hardly a conservative one.

The most serious objection to the proposed method of water-proofing is that it fails of its purpose at the most critical plane—the base of the dam. It is on this plane that the majority of failures may be expected. The author proposes to use a cut-off wall with a water-proofing membrane within it. Presumably the cut-off wall is to be carried down to intercept the upper surface of impervious rock; otherwise leakage would find its way under the cut-off wall and exert its upward pressure on all parts of the base. The excavation of a trench for this purpose will often impair the stability of a dam by breaking up strata of rock which, while allowing leakage would, if undisturbed, be capable of sustaining the dam. The excavation for the cut-off wall would usually be very expensive by reason of the extraordinary care needed to avoid shattering adjacent rock to the extent that it too would have to be removed, thus resulting in a general deepening of the foundation and, therefore, an increase in the height of the dam and the pressure acting on it.

To free a dam from the menace of upward pressure by water-proofing might thus result in a marked increase of cost, which could easily more than offset the savings claimed, even if it were certain that the stratum intercepted was absolutely impervious, but the number of dam sites that will afford rock warranting that much confidence is indeed small. The usual condition will be that in spite of all cut-off walls, however deep and well water-proofed, upward pressure on the base may be expected, due to the presence of seams capable of conducting the water under the wall and to all parts of the base. It seems far better to recognize this danger to the dam's stability and care

* Maj., Corps of Engrs., U. S. Army, New Orleans, La.

for it by providing suitable sub-drainage and a sufficient weight of masonry to resist all forces.

The difficulties that would arise during construction due to the use of the proposed membrane should be mentioned. These difficulties include the support of the membrane during the depositing and spading of concrete within the form, because of the area being broken up by the membrane and whatever appliances are used to support it. In the 3-ft. space up stream from the membrane the placing of concrete would be slow and tedious. Care would have to be taken that this space was not filled faster or slower than the remaining area within the form; otherwise, the pressure of the green concrete would displace the membrane, unless it were rigidly supported.

Concrete dams of considerable height are usually erected in sections from 30 to 40 ft. long and each section in "lifts" of not more than 12 ft. because of the difficulty and expense of installing forms that will support the pressure occurring when concrete is poured continuously to greater heights. This process results in approximately horizontal planes of weakness in each section, to which water may penetrate and on which sliding or overturning of the upper part of the structure is conceivable. It is easily possible, however, to treat these construction planes in such a way as to obviate any danger of failure. The surface obtained at the conclusion of a pour can be roughened by moulding troughs into which the concrete of the next lift above will be poured, and by scrubbing the new surface with wire brooms so as to remove laitance and expose portions of the aggregate; and bond may be obtained by driving reinforcing steel several feet down into the concrete leaving the upper ends protruding so as to hold down the next layer when poured. All these precautions can be effected by the use of a fair-sized labor gang in each form for a few hours just after the pour is completed and without in any way interfering with the pouring of the concrete or delaying future construction, but they can not be depended on to prevent all leakage and the pressure resulting from it. However, leakage should not occur elsewhere than at the horizontal and vertical joints if the concrete is properly proportioned, mixed, and placed.

The writer was connected with the construction of a dam nearly 1 mile long and impounding water to a height of about 95 ft. It was constructed in sections, each from 30 to 38 ft. long in the direction of the long dimension of the dam, and the blocks were built by pouring successive layers of concrete, usually 6½ ft. thick, one above the other. The top of each layer was roughened by moulding grooves therein into which the concrete of the next layer above protruded; the whole surface was scrubbed to remove laitance and expose portions of the stones composing the aggregate; and a thin layer of neat cement grout was applied before the next layer of concrete was placed. Notwithstanding these precautions, a number of these seams, probably less than one-quarter of the total number in the dam, allowed sufficient leakage to visibly moisten the down-stream face of the dam. In no case did the water issue in streams. The upward pressure produced by this leakage is unknown. It is conceivable that the water pene-

trating into these horizontal seams, however small in quantity, is capable of developing an upward pressure corresponding to a substantial proportion of the static head.

In the dam in question, the possibility of such pressures caused no apprehension, because the thickness of the dam at any plane was fixed largely by the width of the base and the requirement that the down-stream face be shaped to act as a spillway on occasions. When designed to meet these requirements, ample stability was obtained even considering all possible upward pressure, and, therefore, no saving of materials could have been made by water-proofing the horizontal joints. The existence of such leakage is objectionable, however, because it is unsightly, induces doubts as to the quality of workmanship, tends to impair public confidence in the stability of the structure, and results in losses of water, however small, which the dam is supposed to conserve. Various water-proofing fabrics impregnated with bituminous, or asphaltic compounds, or such compounds rolled out as sheets, are available. It is believed that narrow sheets of such substances or of metal can be placed so as to span the joints between pours without the labor and expense incident to the use of the continuous membrane proposed by the author and that such treatment of the joints will prevent visible leakage.

The treatment of the vertical joints is more complicated. In this case, there is no overlying weight tending to close the joint and the width of the joints varies with the temperature of the concrete. Leaks through vertical seams will generally be larger than those through horizontal seams. In the dam previously mentioned, the leaks occurring through the joints between adjacent construction sections reached considerable proportions, and in several places water streams of considerable size issued freely from the down-stream face of the dam. The inspection tunnel in the lower part of the dam, intersected, of course, by the vertical construction joints, could be kept dry only by the use of much more pumping equipment than had previously been thought necessary. The leakage was of such proportions that elaborate and expensive measures to stop it are in progress.

This dam was built without true expansion joints. Several thicknesses of tar paper were used between adjacent sections at the outer margins of the surfaces in contact, but the main bodies of the sections were in direct contact with each other. After one section had been poured and the side forms removed, the surface against which the adjacent section would be poured was thoroughly cleaned; but as the forms could not be removed while the concrete was green, it was impracticable to roughen the surface by scrubbing in the same way as was done on the horizontal joints. Key boxes were nailed on the inside of the forms, however, thus providing for tongue and groove joints between sections. No membrane of any kind spanned the joints between sections, but the writer now believes that provision for seals of some kind should have been provided. It is believed that in future construction metal or asphalt-treated fabric strips might be folded and tacked to the side forms in such a way that half the width of the strip, say, 8 or 10 in., would be embedded in the concrete as poured and the other half lie flat against the form so that when

the form is removed, it might be straightened out and become embedded in the adjoining section when poured. This type of construction would prevent the bulk of the leakage and, at the same time, be free from the objections to the use of a continuous membrane as proposed by the author.

The text and Fig. 1 of the paper indicate a proposal that the vertical expansion joints be filled with asphalt. While expansion joints may be needed in pavements, bridge floors, and other structures of limited thickness, and can be readily provided in such cases, their use in a massive structure, such as a dam or a thick wall, is not necessary and would lead to extraordinary construction difficulties unless the asphalt were applied cold in sheets to the side of one section prior to the pouring of the next section against it. In this case, the asphalt sheets would obtain less intimate contact with the concrete than if it were poured hot into the spaces between the sections. Space for the filling is easily obtained in floors, sidewalks, and streets by withdrawing the form between adjacent blocks before the final set of the concrete; but this method is not applicable to heavy walls or dams because of the difficulty of forming the narrow spaces between the heavy sections. Any template or form left in the concrete for this purpose would be gripped so tightly by the concrete that its removal would be very difficult if not impossible. The use of a plastic material, such as asphalt, in such joints does not always serve to seal the joint so as to prevent leakage. The writer has noted in many joints so treated that in cold weather when the joints open the greatest amount, the filling becomes brittle, breaks away from one of the surfaces of the joint, and clings to the other, thus failing to close the opening. This applies to the filling as ordinarily used in expansion joints. Better results could probably be obtained by placing strips of metal or water-proof fabric so as to be perpendicular to the plane of the joint, with approximately half the width of the strip embedded in each section. Some of the materials that could be used for this purpose are sufficiently elastic to accommodate themselves to the opening and closing of the joints due to temperature changes, but if non-elastic strips are used they can be readily crimped, somewhat in the manner shown by the author in the enlarged view of the joint at Section *A-A*, Fig. 1, thus permitting all possible variations in the width of the joint without over-straining the strip.

The writer concurs with the author as to the desirability of water-proofing a dam especially at the horizontal and vertical joints, but doubts that such measures can be made so effective as to warrant the omission of uplift from consideration in calculating the dimensions of a dam. Such savings as could be made would be of no use unless the base could be water-proofed so as to decrease substantially the required weight of masonry above the base. The use of continuous membranes as proposed appears unnecessarily complicated and could well be abandoned in favor of narrow strips or sheets placed so as to span the joints.

K. E. Hilgard,* M. Am. Soc. C. E. (by letter).—The interesting and important topics of the porosity of masonry and concrete dams, together with

* Cons. Engr., Zurich, Switzerland.

the hydrostatic pressure in seams, the fissures or more or less open joints under and in such structures, and the consequent uplift created, have long since led to occasionally serious discussions and quite contrasting views, as well as to widely differing precautionary measures.

In the author's brief history of dam design it might be recalled that the consideration of uplift in and under masonry dams was first proposed in the United States by the late James B. Francis, Past-President, Am. Soc. C. E., in 1888.* He assumed the full hydrostatic pressure in the bed-joint at the base at the heel, uniformly diminishing to zero at the toe. Shortly afterward (in 1889) and quite independently, Mr. Lieckfeldt, using a different assumption, namely, that the hydrostatic pressure in every bed-joint could only exert an uplift to the extent that it exceeded the pressure from the superimposed weight of the masonry, developed and published a complete method of computation.† In the same year, Mr. H. Fecht, who had built or was building several masonry dams in the Vosges Mountains (Alsace), quite independently of the other two engineers and their assumptions reached the same conclusion as Mr. Francis, applying it to all bed-joints throughout the dam. According to his theory each layer of masonry confined between two bed-joints is subject only to an uplifting force equal to the difference between the hydrostatic downward pressure in the upper joint and the upward pressure in the lower joint, both acting on this interjacent volume of masonry.‡

The designs of the two highest gravity masonry (concrete) dams in Switzerland ("Barberine" and "Waeggital"), the former slightly curved and the latter built rectilinear in plan, and both completed in 1925, were based (by approaching French regulations) on the assumption of 80% of the full hydrostatic head on the up-stream side diminishing uniformly to zero. By observations the engineers were convinced that, with very careful workmanship and subsequent numerous injections of cement at the base and into the underlying rock, in reality little uplift could take effect. The two dams have maximum heights of 266 and 366 ft., and the crowns are at an altitude of 6 300 and 3 000 ft. above sea level, respectively.

Neither dam was specially coated on the up-stream face, but instead a mix richer in cement was used there. The careful proportioning resulting in a maximum of water-tightness was previously determined by experiments on very large and thick concrete disks, which were tested as to their impermeability by water under high pressure. Most of these tests were made under the supervision of a Special Commission of the "Swiss Association for Economic Conservation of Water". In both structures a high degree of water-tightness has been obtained. No interior drainage proper was provided in either dam.

A number of dams have been built in Germany from designs of Professor Dr. Intze and others to prevent uplift in the interior of the masonry. Professor Intze rather insisted on the possibility of producing water-tight masonry with proper material, good workmanship, and a careful coating of

* *Transactions*, Am. Soc. C. E., Vol. XIX (1888), p. 159.
† *Centralblatt der Bauverwaltung* (Berlin), 1889, pp 397 and 443.
‡ *Zeitschrift für Bauwesen*, 1889, and "Thalsperrenbau," P. Zeigler, Berlin, 1900, p. 144.

the up-stream face; but at Solingen, in the Urft Valley (Eifel Mountains), and at Marklissa, a curtain of vertical drains of porous tile or perforated glazed pipes of 2 to 3 in. clear width was used, spaced about 8 ft. apart throughout the entire height of the up-stream face and 5 ft. from the surface. These emptied into drainage—or inspection—tunnels and thence into a shaft.*

Experiments in Switzerland carried on by the Special Commission mentioned previously and also by a Special Commission of the "Swiss Engineers' Society", have shown that the correct proportioning of cement, water, sand, and gravel, with careful placing of the mixture, is far more effective in producing water-tight concrete than an admixture of any of the many ingredients widely advertised for that purpose.†

As to the means of creating an absolutely water-tight curtain in the interior of the dam by inserting a totally flat membrane of sheet zinc, lead, or copper, as shown in Fig. 1, it is feared or at least considered possible that temperature and moisture on the up-stream side might cause a separation between the membrane and the concrete. As this proposal has not been actually tested, such a test would seem to be highly desirable. It should be extended to an insertion of a differently shaped membrane consisting of a series of corrugations similar to the one shown in Fig. 4 of Mr. Gardiner's discussion. This form is now manufactured commercially of sheet iron, but could just as well be produced of zinc, lead, or copper, with soldered or electrically welded joints.

By applying more mortar than coarse concrete near the membrane all interstices could be securely filled, thus obtaining an interlocking contact between both sides of the membrane without anchors.

Thomas H. Wiggin,* M. Am. Soc. C. E.—The author develops an idea that is worthy of careful examination. For a great many years the importance of excluding water pressure as far as possible from joints in and beneath gravity dams has been realized by designers; but hitherto the preventive steps taken in construction have been (*a*) to grout seams in the foundation; (*b*) to carry a cut-off wall down in a trench below the general level of the dam; (*c*) to strive for dense masonry with joints filled as completely as practicable; and, in a few cases, (*d*) to provide drainage near the up-stream face of the dam.

In repair work after excessive seepage has occurred, various schemes involving caulking, gunite, and an additional layer of masonry, have been used to try to correct defects. A recent case of this kind is the Delta Dam at Rome, N. Y., a part of the water supply system of the Barge Canal. Here a covering of rich concrete masonry from 1 ft. to 4 ft. thick has been used under specifications with which the speaker was informally concerned. This type of construction was adopted after considering methods similar to those suggested by the author.

* Handbuch der Ingenieurwissenschaften, 1913, 11 Bd., 2 Abt, "Die Talsperren," by E. Mattern.

† "Gussbeton," Erfahrungen beim Schweiz. Talsperrenbau, S. I. A. Verein, Zurich, 1926 (In pamphlet.)

‡ Cons. Engr., New York, N. Y.

Men of two schools of thought are designing dams, those of one school assuming that provisions like those already listed will be successful in preventing uplift water pressure and hence adding no weight in the design on account of uplift; and those of the other school adding weight as well as taking the precautions to reduce uplift pressure. It is impossible to know what proportion of dams has the extra weight to provide for uplift pressure (on some assumption) and what proportion has not. In general, the older dams lack the provision and some of the recent dams follow the older method, especially where economy is thought to be important. Success of the older dams would be justification for the lighter designs but for the unfortunate fact that a comparatively large number of gravity dams have failed, although the causes are not usually easily analyzed. Beyond doubt, however, sheer additional weight would have helped them.

Assuming it to be practicable, by use of water-proofing such as the author proposes, to eliminate all doubt as to uplift pressure in the masonry, there still remains the more difficult questions of junction between the dam and its foundation as well as of joints in the rock immediately beneath the dam. Saving in weight in the dam would be impossible unless these lower and most stressed joints are also water-proof and can be absolutely proved to be water-proof. A liberal number of drainage holes drilled so as to communicate from the down-stream side of the supposed cut-off to visible outlets at the toe of the dam might prove the condition fairly well but only after the dam were in service, when it would ordinarily be too late to use the information, if adverse.

If one could be confident of making the joints beneath the dam tight, then the doubt as to tightness of joints in the masonry above might be resolved by the use of vertical reinforcing steel on the up-stream face of the dam.

The successful design and construction of a permanently water-proof membrane is not an assured matter. The speaker has had a number of experiences in this line. One of the tunnels constructed by the Board of Water Supply of New York City became damaged by a widespread disintegration of the concrete lining where the tunnel passed through rock containing pyrite. A thin sheet steel lining to conduct off the water which filtered into the tunnel was placed inside the concrete lining, and within that a 3-ply membrane of asphalt and impregnated cotton drilling. A brick lining, 12 in. thick, was then placed within the water-proofing.

Every precaution was taken to make the membrane water-tight. Wherever it was necessary to put a grout pipe through it, interlapped fabric patches were used and compressed between flanges screwed to the grout pipes. Afterward the rock, the space between the rock and the concrete lining, that between the concrete and the water-proofing, and that between the water-proofing and the brickwork were grouted very carefully with minute attention to sequence of work and pressures. The job was successful, practically speaking, but there was plenty of evidence that water got through some of the joints or pipe holes in the water-proofing.

Recently under the speaker's direction the sloping deck of a multiple-arch dam was water-proofed, the lower parts with 3-ply, and the upper parts with

2-ply, impregnated cotton drilling and asphalt. The conditions were ideal for perfect work, the slope of the dam being convenient, the work being all in daylight, and no anchor-bolts piercing the water-proofing. A skillful and experienced superintendent was in charge and day labor was used. Again, there were some slight signs of penetration of water. It is not known to what extent the water travels laterally between the arch and the water-proofing.

There is another matter, namely, the question of freezing, which might be important in a northern climate. If water were to soak in against the water-proofing there would be a tendency for a film of water to gather between the outside jacket of masonry and the water-proofing; a gradual heaving of the outside jacket might occur progressively.

Quite apart from the saving in weight, the water-proofing, if successful, would protect the down-stream face of the dam from disintegration which too often happens, apparently owing to the freezing of seepage water on concrete that seems frequently to be all too susceptible to such damage. It was with the idea of protecting both faces of the concrete from the effect of freezing moisture that the multiple-arch dam previously mentioned was water-proofed.

The dovetail method (Fig. 4), used by Mr. Gardiner in locking concrete together across a water-proof membrane would seem more promising than a multiplicity of bolts piercing the membrane.

An adequate factor of safety in gravity dams is most important. If water-proofing could be proved to be effective in preventing uplift pressure which it was known would otherwise actually exist, then water-proofing would have the effect either of making a lighter dam equally safe or of increasing the factor of safety for a dam of the same weight. If, however, the uplift pressure does not actually exist to an important degree in dams that are not water-proofed, the practical effect of water-proofing a dam and omitting allowance for uplift pressure would be to reduce the factor of safety which already exists in the more conservative designs. There has been more tendency among designers of dams than among designers of steel bridges, for example, to design on the "ragged edge" of safety. The factor of safety chosen for a dam will be affected, of course, in some measure according to the size of the flood that would be let loose by a failure and the number of people and value of property down stream from the dam. The failure of most dams of considerable size, however, would result in some deaths and the world can well afford better protection than has been provided in too many cases for all those obliged to dwell or to work down stream from such structures.

Jerome W. Howe,* Assoc. M. Am. Soc. C. E. (by letter).—In any current consideration of this subject it is pertinent to take stock of the study devoted to it rather recently by a French Government Commission, and the conclusions respecting it embodied in the report† to the Ministry of Public Works.

* Prof., Civ. Eng., Worcester Polytechnic Inst., Worcester, Mass.

† "Barrages de grande hauteur, résistant par leur propre poids": Instruction pour la préparation des projets et l'exécution de travaux adressée à MM. les Ingénieurs en Chef des Ponts et Chaussées par M. le Ministre des Travaux Publics (Circul. Série B, No. 67, 19 octobre 1923). Published in *Annales des Ponts et Chaussées*, Vol. 11e, Série 93e, Année No. 30.

The author pays due respect to the memory of de Sazilly and Delôcre, with whom the modern rational methods of dam design and analysis orginated. Subsequently, leadership in this field of engineering passed to the United States, by virtue of the far greater extent and magnitude of masonry dam construction here, as contrasted with that in other parts of the world, even admitting the importance and excellence of a large number of high masonry dams in Germany, Australia, and elsewhere. Latterly, however, the increase of water-power development in France has caused French engineers to focus attention again on the masonry dam, with a view to bringing about greater economy in dam construction, and thus contributing to the advance of the National water-power development. It will probably be of advantage to American engineers to keep informed of the trend of French engineering thought respecting this important subject.

The Commission of Engineers referred to did not essay any especially original approaches to its problem. It took what was probably the wiser course of studying the most advanced practice, particularly as exemplified in American dams. As a carefully considered judgment based on the whole history of the construction (and destruction) of masonry dams, and more particularly on a survey of the history of American practice, the several items of the report merit attention. In this connection, only that part of the study concerned with the matter of hydrostatic uplift is relevant. It should be remarked in passing, however, that several items of the report appear to have considerable value, particularly an appendix explaining M. Pigeaud's method of calculation, and the method of plotting results in curves of equal maximum pressure and equal maximum shear, in isostatic lines and lines of sliding.

The failure of the dam of Habra in 1881, and that of Bouzey in 1895, so shortly after the application of rational principles had been attempted for the first time in connection with the Furens Dam, caused general suspicion of the rational methods of design. Maurice Lévy, however, assigned to these failures the probable true cause—hydrostatic uplift; and he recommended interior drainage systems. He seems to have intended, rather, exterior drainage systems, for his recommendation was, in substance, to provide a protective wall separated by means of pilasters from the up-stream face. Except for possible poor bond under the influence of temperature stresses and unequal settlement, this was in effect a provision for a system of wells close behind the up-stream face, for which an under-drainage connecting canal was a necessary adjunct. Many French dams have been thus constructed in recent times.

The Commission finds that this provision has added about 30% to the volume of masonry and, consequently, to the costs, and is not favorable to a continuance of such practice.

In its consideration of hydrostatic uplift, the Commission reviews the Italian practice, which is to calculate such uplift in most cases, and to use a unit pressure varying uniformly from $m \gamma y$ at the up-stream face to zero at the down-stream face, γ being the unit weight of water; y, the depth below water level; and m, a constant varying with conditions from one-third to unity. This constant becomes equal to zero only in low dams with most favorable con-

ditions (especially as to foundations and materials of construction). However, the French Commission recommends the design of most masonry dams on the basis of no hydrostatic uplift.

The Commission declares itself in favor of the American practice, exemplified in the Kensico Dam, of a network of drainage wells and galleries near the up-stream face, with perhaps a second series toward the center of the structure. In many dams the wells are too small to permit of inspection. The best practice is that adopted at Kensico, of wells 1 m. square spaced (in this instance) 85 ft. apart, with smaller wells (in this instance, six) in each interval, the latter lined with porous concrete blocks.

Respecting methods of making the up-stream face impermeable, the Commission notes the successful application of coatings in a number of instances, particularly coatings of asphaltic materials, and, as at the Elephant Butte Dam, gunite. It gives special emphasis, however, to the possibility of making the masonry of the up-stream face itself impermeable, or practically so. In this connection, also, reference is made to the Kensico Dam, which was built, as was also the Olive Bridge Dam, with a facing of pre-cast concrete blocks made with special care and placed with thin joints. The Commission's favorable reference to Kensico is well justified. A visit to the inspection galleries reveals a discharge into the gutters of the lower inspection gallery of not more than about 7 000 or 8 000 gal. per day, most of this being from the expansion joints. Very little leakage is contributed from the seepage wells.

The Commission is especially favorable to the practice of obtaining practical impermeability by enriching the cement content of the concrete for a distance back from the face varying with the water pressure. It refers to the Arrowrock and Hetch-Hetchy Dams as instances. In the former structure the mix was 1: 9 for a thickness of from 5 to 10 ft., the remainder of the structure being poured with a 1 : 10.5 mix. At Hetch Hetchy the mix of the up-stream face was 1 : 7.5 (instead of 1 : 9 as for the mass of the structure) for a thickness of about 4 or 5 ft. Attention is called, however, to the increase in cost resulting from such a practice, and the recommendation is made that instead of securing impermeability through excess of cement over that required for strength and durability, the recent practice of the U. S. Reclamation Service of using sand cement be followed. This sand cement is obtained by grinding to a high degree of fineness a mixture of cement clinker and pebbles of hard stone (such as granite, sandstone, basalt). At the end of several years, it gives, so it is stated, a strength at least equal, sometimes even superior, to that obtained with pure cement, and it always gives a greater impermeability. Its use permits of varying the strength from bottom to top of the dam, without at the same time diminishing the imperviousness of the face. Scientific proportioning of the concrete, by the "water-cement-ratio" principle, with careful grading of the aggregates and control of the water, are also counted upon heavily to achieve a practically impervious concrete.

To prevent the under-passage of water, sheet-piling, pressure grouting of the foundation area, drainage of the foundation just down stream from the sheet-piling, and a cut-off wall of rich concrete are recommended. Foundation

drains under the up-stream part of the dam should be pierced after the injection under pressure of grout into the seams of the sub-rock. They should lead to the inspection gallery within the dam, so that their discharge can be observed. In case of excessive discharge, additional pressure injections can be effected, using for this purpose the inspection gallery and drainage wells. At the Elephant Butte Dam 360 tons of cement were injected into the foundation rock. A double row of foundation drains was constructed. That nearest the up-stream face lies at a depth of 45 ft. below the foundation; that nearest the down-stream face is only a few feet below the foundation.

To summarize, the conclusion of this Commission was that it is usually possible by taking proper precautions (galleries, drains, surfacings, cut-off walls, pressure injections, etc.) and by meticulous care in the construction, to avoid the penetration of water under pressure either into the masonry of the dam or through the foundation rock in the vicinity of its base.

W. H. R. NIMMO,* ASSOC. M. AM. SOC. C. E. (by letter).—The allowance to be made for uplift under masonry dams, as recommended by various designers, ranges from full hydrostatic head over the base to zero. The author assumes that for masonry dams as now constructed there will be water pressure under the base varying uniformly from two-thirds full hydrostatic head at the up-stream face to zero at the down-stream face; and he then shows that if this uplift pressure is entirely prevented by a water-tight diaphragm placed near the up-stream face the saving in material in the case of a high dam will amount to about 14 per cent.

Such a diaphragm, if adopted, would have to be placed sufficiently far back from the face of the dam to be well beyond the influence of freezing. Where uplift pressures of the intensity anticipated by the author cannot be prevented by other means a metal diaphragm may be well worth considering. For a dam on a porous foundation it is necessary to assume that the uplift pressure at the up-stream face will be equal to the full hydrostatic head, but for a well designed and carefully constructed dam on a reasonably sound rock foundation an allowance of even $\frac{2}{3}H$ seems to the writer to be excessive. If care is taken to avoid shattering of the foundation rock by the use of heavy charges of explosives, and a cut-off wall is constructed with a system of drains behind it discharging into an inspection tunnel, then only small pressures, if any, should be developed in the foundation.

The body of a high and important gravity dam should also be protected by an efficient drainage system consisting of approximately vertical drains, at frequent intervals, running in straight lines from an upper to a lower inspection gallery. All drains should be large enough to be properly cleaned. If a leak develops under the base it will be detected by the drains and can probably be stopped by boring holes and grouting under pressure. If the dam is composed of concrete a relatively richer mix may be used in the immediate vicinity of the up-stream face. If the dam is to be filled rapidly it may be advisable

* Chf. Draftsman, Main Roads Comm., Brisbane, Queensland, Australia.

to plaster the water face with a cement gun, since, although this coating may not be permanent, the concrete is thereby protected from intrusion of water until it has become thoroughly seasoned. If these precautions are taken, only a small allowance for uplift should be necessary.

A commission of French engineers, reporting on the design and construction of high gravity dams, expressed the opinion that it is usually possible to prevent the penetration of water under pressure either into the body of the dam itself or into the foundation; and that, except under special circumstances, uplift should not be taken into account in designing a gravity dam.*

An arched dam, recently built across the Jogne River, in Switzerland, and having a height of approximately 195 ft., was provided with drains from the foundation. Regular observation after the filling of the reservoir failed to disclose any flow of water from the drains.† If it be conceded that the author's two-thirds rule should be generally adopted, an arched dam the stability of which is not menaced by uplift pressure, has a great advantage over a gravity dam in this respect.

E. W. Lane,‡ M. Am. Soc. C. E. (by letter).—There seems to be enough saving in cost in the method of dam construction proposed by Mr. Pagon to make it worthy of serious consideration in some cases, but it should be used with caution. Any reduction in the volume of masonry in a dam means a decrease in weight and, therefore, a smaller factor of safety against sliding. In many cases the danger of sliding is much greater than that of over-turning even with uplift neglected.

The writer does not recall a single instance of a dam which had a resultant pressure within the middle third, neglecting uplift, that failed from over-turning (with the possible exception of cases due to vacuum effects in over-fall dams); but several with such a section, or even a heavier one, have failed from sliding. At many sites the requirements for stability against sliding, therefore, may preclude the use of Mr. Pagon's suggestion.

The necessity of considering uplift in the bed-rock is important even if it is considered in the design of the cross-section of the dam, especially if the foundation is a rock with strata horizontal or dipping slightly down stream. In such cases the writer believes the foundation layers should be grouted or drained or, better still, both; although in some cases the cut-off with a diaphragm as suggested by Mr. Pagon might be preferable.

Unless the seepage is cut off, or vented, to prevent the building up of pressures in the bed-rock, the usual allowance for upward pressure may be insufficient. Consider a dam founded on an impervious rock layer, below which is a seam; for some distance down stream from the dam the rock surface slopes away, as in Fig. 5 (*a*). If the water stored above the dam enters the seam near the dam, and, decreasing uniformly in pressure, escapes at the lower end of the seam, the uplift on the dam and layer may be so much greater than that usually assumed in dam design that the factor of safety

* Ministère des Travaux Publics, Circulaire Série B, No. 67, Paris, October 19, 1923.

† "La Construction des Barrages-Voutes en Suisse," by H. E. Gruner, *Le Genie Civil*, August 29, 1925.

‡ Hydr. Engr., Mississippi River Spillway Board, U. S. War Dept., New Orleans, La.

against sliding, even considering the extra weight of the rock layer, is much less than that estimated. Such a condition may have caused the sliding of the dams at Austin, Tex., and Austin, Pa. In case the strata slope down, as in Fig. 5 (*b*), the pervious joint at which the water escapes may be so far from the dam that practically the full upward pressure is effective over the entire base, although the weight of the rock between the seam and the bottom of the dam would partly offset this. A dam with such a foundation could not readily slide, but the factor of safety against over-turning would be reduced.

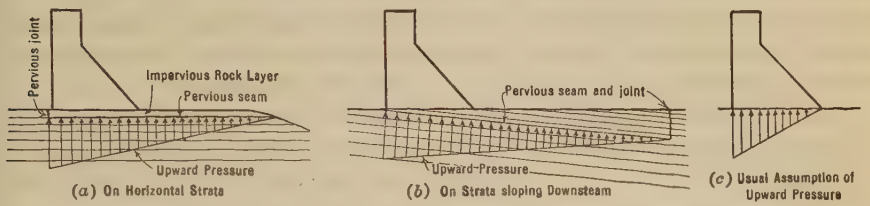

FIG. 5.—VARIOUS CONDITIONS OF UPWARD PRESSURE UNDER DAMS.

For conditions which permit the use of the method proposed by Mr. Pagon the same result as regards stability of the dam proper against over-turning could be more cheaply accomplished in a concrete dam by adding tension reinforcement near the up-stream face to resist the over-turning effect of the upward pressure. It would be necessary to take precautions to have the rods well protected against corrosion at the horizontal construction joints, but this could be easily done. If necessary, the tension rods could be encased in a precast unit before being placed in the dam. Many dams are now built in which the entire reliance against failure is placed on the protection of a few inches of concrete, and it should not seem very radical, therefore, when five or ten times that thickness is used to insure an effect which may not be necessary for stability. The writer does not wish to advocate this form of construction as he has not given the subject sufficient study, but the possibility would seem worth consideration. This method, however, would not prevent seepage through the dam as would a diaphragm.

NATHAN C. JOHNSON,* ASSOC. M. AM. SOC. C. E.—Since the author's water-proofing membrane is proposed *per se* as a means to prevent uplift in dams by stopping the entrance of water into the concrete or masonry, is the situation not this: Assume a more or less hollow structure, of ample weight for stability, but made, say, of sheet iron so as to be impervious. There will be no uplift unless the structure, instead of being filled with water, is filled with air. There is, therefore, no uplift to be anticipated for any more or less hollow structure, but only for a structure from which water is excluded.

In a concrete dam there would be no uplift except through voids filled with air which are in a very small percentage, assuming that no water enters under the dam.

This viewpoint is taken in order to divorce the subject of uplift from that of concrete as a material. This latter is a separate subject, particularly as regards its abilities or disabilities for use in water-exposed structures.

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The discussion of this paper, therefore, calls for division into (1) the question of entrance of water *versus* uplift in any water-retaining structure; (2) the actuality, magnitude, and distribution of forces exerted by uplift; and (3) concrete as a material suitable for water-bearing structures and the necessity, or lack of necessity, for the water-proofing concrete because of its inherent disabilities, or because of the result commonly produced in commercial work.

By its very nature concrete is never free from water, but it should not pass water in sensible quantities. If concrete is free from water, it is a disintegrating and more or less pulverulent mass. Concrete must have a normal water content to maintain its integrity; and any proposal to exclude water must be carefully evaluated before it can be pronounced advisable; and a clear differentiation should be made between the exclusion of water and the adoption of means to prevent percolation of water through the mass because, perhaps, of anticipation that a poor job will be had in the actual construction of the barrier.

As to concrete as it is commonly made and the prevention of percolating water, many means have been proposed to secure immunity from the evil of poor work, which is far too prevalent.

When Mr. Pagon proposes a water-proofing membrane for concrete dams, the speaker can see some values as a temporary preventive of first-hand leakage, but no preventive of either permanent proofness or of uplift.

In concrete construction, what is needed first of all is a more general appreciation as to the true nature of concrete, one that will be on a plane with the artistry of which draftsmen are capable in their designing. The draftsman's picture-section of concrete shows a uniform, homogeneous substance, while the product in the field is a highly variable and non-uniform substance and through this field-produced article, water passes in billions of gallons daily. On that rock, practice and theory are split asunder; and much of the confusion of thought that exists to-day arises from that one cause.

In conclusion, therefore, it seems to the speaker that Mr. Pagon should divorce the subject of uplift from the subject of water-proofing; that he should divorce the subject of concrete from the subject of uplift; and that his conception of a water-proof membrane in the face of the dam as preventing uplift is a misconception and unrelated to the subject of uplift *per se,* or to the designing of dams for either safety or economy.

F. W. SCHEIDENHELM,* M. AM. SOC. C. E.—The speaker finds himself thoroughly sympathetic with the desideratum, advocated by the author, of water-proof membranes to be located near and approximately parallel to the up-stream faces of masonry dams. In the reason for this sympathy, however, he differs somewhat from the author, who emphasizes probability of diminution in cross-section of a dam equipped with such a membrane and of consequent reduction in cost. The speaker, on the other hand, is attracted by the hope that such a membrane might constitute an important contribution toward the permanence of the masonry, especially if that be concrete.

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In the paper much stress seems to be laid on the effect of hydrostatic uplift pressure, particularly within the body of a dam, in determining the cross-section and hence the economy of the structure. However, the speaker questions whether such interior uplift pressure is really so important as the paper implies. For instance, he has no direct personal knowledge of any failure of an upper portion of a masonry dam by over-turning or sliding on a lower portion except where caused by ice thrust. Only a single well-authenticated case of failure of a dam even by over-turning on its foundation bed other than due to ice pressure has come to his attention and that case is stated to have involved a dam only 20 to 30 ft. in height and of obviously too thin a cross-section.

It is true that uplift pressure in effect reduces the net weight of a dam or, more accurately, may reduce the load and hence frictional resistance on a possible plane of sliding, and that in this way uplift pressure may contribute to that most important form of failure of masonry dams, namely, failure by sliding. Even so, the uplift pressures that are most dangerous are those existing not within but under a dam, either between the masonry and the natural foundation bed or between approximately horizontal strata of the foundation bed itself. The author implies that he would utilize only such foundation beds as are "tight". However, if he can avoid seamy or porous foundation beds he is singularly fortunate. More often the problem is one of recognizing a seamy or porous condition of foundation bed and of so designing the dam and its appurtenances as to make the resulting structure stable.

The failure of the dam at Austin, Pa., has been mentioned. Although uplift pressure may have been one factor, yet it was not necessarily the only factor and the failure was primarily one by sliding. Certain portions of a concrete cut-off wall, such as it was, remained embedded in the foundation rock—of partly disintegrated, argillaceous sandstone with clayey interbedings. There must have been cracking or complete rupture of that concrete cut-off before any material uplift pressure could have been exerted under the body of the dam. Slender and inadequate as was the cross-section of the dam, the deficiencies in that respect were hardly so serious as those involved in the provision against sliding. The dam might have slid even if there had been no uplift whatever.

There are various expedients for minimizing uplift pressures under a dam. First, there is the expedient of constructing a cut-off in which, as the author points out, it is possible to embody a membrane intended to be water-proof. (However, inasmuch as the concrete of a cut-off wall is not subject to alternate wetting and drying, or freezing and thawing, the speaker would not feel greatly concerned about such percolation through a cut-off as might occur in the absence of a membrane). The cut-off itself, whether it be of concrete or of sheet-piling, may be extended downward still farther by means of pressure grouting with cement, bituminous compounds, etc.

Against the contingency that the cut-off provision may not extend sufficiently deep, or for any other reason may not be absolute, there is possible a second line of defense, namely, a row of deep drain holes, relieved at the top by suitable means to tail-water pressure and extending downward as far

as may be necessary. If, for instance, the base of a dam is 100 ft. wide, the line of such drains might well be 10 to 20 ft. down stream from the up-stream face of the dam and parallel thereto. Surely if these drain holes are spaced closely enough, they would be adequate to prevent any uplift pressure substantially in excess of that due to tail-water. Some might feel concerned that in course of time these drain holes would become clogged, but uplift pressure will presumably be as severe initially as at any time in the life of a dam and even a slight differential pressure will maintain an upward flow and hence relieve the pressure through the drain holes.

In short, it would seem that along the line of such a barrage of drain holes one may reasonably expect the intensity of uplift pressure to approximate that due to tail-water. Whether the maximum intensity of uplift pressure at the up-stream face of the dam be equal to 100% of that due to head-water, or whether it be some fraction thereof, such as two-thirds, is not, under such circumstances, of major consequence. Moreover, such uplift pressure as does exist can not obtain over the entire base of the dam; otherwise the dam would be afloat. Against encroachment of hydrostatic pressure from ground-water or of reservoir pressure coming in from the hillsides, one may similarly set up a defense by extending lines of drains down stream from and at right angles to the main line of drains, such auxiliary lines, of course, being located close to the valley banks or perhaps at the ends of the dam.

The scheme of utilizing such a barrage of drains parallel with the axis of the dam and relying on it to reduce to tail-water pressure any uplift under the dam was utilized by the speaker in 1913 in the case of the Cheat River Dam (on Cheat River in West Virginia near the Pennsylvania State line). A year or two later it was similarly utilized in the reconstruction of the Stony River Dam.* More recently, it is believed, the scheme was used in the design of the Wilson Dam at Muscle Shoals.

The now commonly accepted use within the body of a dam of vertical drains and horizontal galleries intercepting the drains is referred to by the author. Apparently, he would consider them much as safeguards or "detectives" (as to deficiencies in the membrane) rather than as assurances that hydrostatic pressures within the dam will definitely be relieved along the line of such drains. The speaker is inclined to the view that there is more danger of clogging of drains within the body of a concrete dam, by deposit of material leached out of the concrete, than there is of clogging of deep drains in the foundation bed. On the other hand, the clogging of the drains in the body of a structure is believed to be of materially less consequence as concerns stability.

As to the economy which might result from the use of a water-proof membrane, such as that of which the author is a proponent, it should be borne in mind that the cross-section and hence the quantities involved in a masonry dam are frequently, if indeed not generally, determined by other considerations. Reference has already been made to the importance of sta-

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), pp. 907-1100.

bility against sliding. In addition, there is the necessity for a suitable "fattening" of the cross-section when it has to serve as an ogee-form spillway. Because of such considerations and of the means available for minimizing uplift pressures, especially under the base, by drainage, a water-proof membrane can hardly be important in effecting economy.

Nevertheless, there is a good reason for giving very serious consideration to the use of such a membrane. In case it were to insure the permanence of the concrete—if only the interior concrete—of a dam built mainly or entirely of concrete, the use of such a membrane would have ample justification. The desideratum would be a means for preventing head-water from percolating or leaking through the interior of a solid masonry dam. Granting a water-tight membrane, one might well afford to pay for the additional up-stream concrete necessary to keep the membrane or diaphragm intact even if such concrete were not relied on in determinations of stability. Evidently, such a membrane would not prevent the weathering or disintegration of the down-stream face of a dam, but it would have sufficient to its credit if it were effective in preventing head-water from penetrating construction joints and the interior concrete of a dam.

Unfortunately, the art appears as yet to offer no well-established case of a membrane that has accomplished such a purpose and has served that purpose for a substantial length of time. A membrane is subject to attack by chemical action and to mechanical difficulties resulting from differences in expansion coefficients. Considerations such as these, even if perhaps not conclusive as to the feasibility of the membrane, leave questionable any design that for stability relies on the permanence of such a layer.

The profession would benefit greatly if there were constructed and demonstrated a membrane for masonry dams that would be water-tight and permanent. Until such demonstration is available, however, the speaker would be inclined to justify the use of "water-proof membranes", if at all, by reason of their probable contribution toward longer life of the masonry rather than by reason of any economies assumed to result from reductions in design stresses and cross-sections on account of reliance on such membranes.

W. Watters Pagon,* M. Am. Soc. C. E. (by letter).—Some misapprehension has been caused by the writer's question "Why build on such a foundation?" This is due apparently to a misconception of his idea. If the foundation is entirely and definitely tight, such as that described by Mr. Wegmann, then uplift need not be included; "if this proves not to be so, then of necessity drainage galleries could not be provided without causing serious leakage", or as will be shown later, a dam built without drainage galleries might develop full hydrostatic pressure under a large part of the base. The intermediate case, then, is one in which by providing a cut-off wall, by grouting seams, or by other means, the amount of leakage can be held within reason, and the presence of drains will relieve all pressure greater than that of the tail-race. If the bed can be made tight, then water-proofing is essential; if

* Cons. Engr., Baltimore, Md.

the bed cannot be made tight, water-proofing to prevent uplift is a waste of money and is of value only as it prevents seepage and the results of seepage.

Mr. Kennison has well emphasized the importance of considering the stress condition in the foundation rock, but does not make clear why this would reduce the savings to be attained. This question also will be discussed later.

Mr. Beatty describes the methods used in one dam to collect seepage. With regard to variation of expansion between a metal membrane and the concrete—a question which he, Mr. Parsons, and others, raise—it has been the writer's thought to permit free movement of the sheet, with mechanical means for localizing it at given points. If Mr. Gardiner's suggestion be followed, no means need be used. With regard to small leaks, the writer has deliberately provided in his design for small drains behind the membrane and larger drainage galleries near the bottom and behind the membrane which are not shown in Fig. 1. With regard to freezing no difficulty should arise through having a membrane, all damage being deliberately localized in the narrow up-stream covering and prevented throughout the body of the dam.

Regarding Mr. Gausmann's comments—the specific saving was noted in only one case, because, regardless of height, the saving depends largely on the profile of the valley and the ratio of the maximum to the weighted average height. Other cases were omitted because the only point of general interest is the saving per foot for various heights, from which it is easy to determine the specific saving for any given case. The fact that Fig. 1 does not show the whole design lays the writer open to Mr. Gausmann's final comment, but actually it has been the intention always to provide drains behind the membrane, as noted on page 241.

Mr. Parsons considers a membrane unadapted to a stone-faced dam; but it is a simple matter to provide near the membrane sufficient concrete to permit of such construction. The cost of forms must, of course, be considered, and was so considered in the single specific case given. The matter of expansion has already been mentioned. If the concrete on the up-stream face of the dam could be made definitely water-tight, with no chance of sporadic cracking due to shrinkage, swelling, and the many other factors that cause it to be constantly changing, there would be no argument for a membrane.

Mr. Flinn mentions seepage through concrete dams and the chance of the drains clogging. The drainage galleries mentioned previously are intended to permit inspection of the smaller drains.

Mr. Gardiner's suggestion is better than the anchorage shown in the paper, provided, of course, that the details and stresses are carefully considered and that the workmanship is suitable.

The writer cannot agree with Mr. Siems that there would be shearing of the protection concrete past the main body of the dam in spite of the anchorage. Mr. Siems mentions the case of dams located above towns or cities in advocacy of a higher factor of safety. The writer believes, on the

other hand, that there should be no change in the true factor of safety to meet conditions; that if in a dam there is any chance of uplift with no definite means of eliminating it, then the weight of the dam should be reduced by the uplift. This, however, does not imply the ostrich practice of burying one's head in the sand and computing uplift merely as a factor of safety. If an extra factor should be deemed advisable then it should be used as such, and not as uplift.

Mr. Jakobsen quotes two dams in which uplift was not considered because means were provided to eliminate it, which is the idea sponsored in the paper. The writer believes it impossible that accumulation of water between the membrane and the face concrete could pry away the concrete facing, because the facing would only be subjected to water loads of equal amount and distribution. Of course a heavier dam will be safer than a lighter one, if both designs have been based on correct understanding of the stresses. Equally, also, the designer should be sure that no tension can exist in the dam; but the inclusion of an arbitrary amount of uplift will not atone for lack of knowledge of the true stresses, nor for failure to search out a more exact method of analysis by which to determine the true stresses. According to customary methods of gravity design it is not the practice to build a 500-ft. dam with a vertical up-stream face; hence the discussion of stresses in such a dam seems inapt.

Mr. Halmos, as well as Messrs. Wegmann, Hilgard, Howe, and others, have made valuable additions to the history of modern dam design. The writer concurs in the belief that tension should not exist, and does not advocate construction that permits it; but should it exist a membrane with a drainage system, as recommended, will certainly reduce the possibility of uplift within the masonry. What to one person may seem a makeshift may to another be an actual advance in the art of designing.

Mr. Wegmann has correctly stated the writer's intent that to be effective a water-proofed dam must be on a foundation that can be made tight to a reasonable depth. Of course, if for the moment the dam and the rock foundation be considered an integral mass, the stress from the dam will spread considerably after passing the plane of juncture by reason of the widening of the rock base, the overturning moment will of necessity increase at a diminishing rate and at sufficient depth become constant due to zero shear, and the net weight of dam and rock less the uplift will continue to increase; hence, the stability on sections below the bed surface will increase with the depth, except in one case, namely, that in which there might exist a water pocket open to full up-stream head and closed down stream so as to prevent issuance of the water, somewhat as shown in Mr. Lane's Fig. 5. In this event there might exist an uplift equal to the weight of a mass of water having a width equal to the width of the bed and a height equal to the up-stream head. Such a mass might well exceed in weight the entire weight of the dam and foundation material, so that the entire mass might float; but so might the entire mass float even if the somewhat customary allowance for uplift had been made. Mr. Wegmann has given an interesting

review of the construction of the dams of the New York water supply especially as concerns uplift.

Major Elliott has rightly called attention to the difficulties of installing a proper membrane. It is true that in certain situations it is better judgment to provide additional masonry rather than to include a membrane. For low heights the writer has shown that there is no economy, and it is not his intent to furnish a cure-all for every situation. Major Elliott's review of the leakage in one particular structure is interesting. That there is often considerable leakage is beyond question, as is also the fact that the base must be properly cared for.

Mr. Wiggin's comments and experience are helpful. The safety of a dam ought to be assured in any case, independent of the number of persons or value of property in the area that would be flooded in the event of failure; but to make allowance for uplift when it cannot exist, or when it can definitely be eliminated, is hardly logical. Is it not better frankly to increase the factor of safety?

Mr. Lane very properly adverts to the question of sliding. The writer is doubtful of the advantage to be gained by anchoring into the foundation rock at the up-stream edge of the base; in one structure he was forced by circumstances to use this method.

Mr. Johnson in his interesting discussion seems to have omitted the case of uplift at a horizontal crack or seam of laitance, hence the writer is not willing to grant the divorce suggested.

Mr. Scheidenhelm questions the existence of uplift within the dam. It is conceded that if uplift cannot be prevented in the foundation bed there is no gain by water-proofing the body of the dam, from the point of view of preventing uplift; there remains only the prevention of seepage. That the writer does not hope always to avoid some leakage in the base is evidenced by his insertion of drains just behind the membrane as already explained; but the dam should not act as a cork, to bottle up all leakage under the foundation bed where it can perhaps develop full up-stream head, when by care in constructing the cut-off, by grouting seams or by other means, the bulk of the leakage can be prevented. Thus, the small hydrostatic pressure that might occur under the bed would be cared for by the drains without causing excessive leakage of water from the reservoir.

The writer appreciates the comments and criticisms of those who have entered this discussion. If as a result there should come about a clearer appreciation of uplift; if in some cases there should result a lowered cost of construction, or, in general, a more thorough investigation of the true stresses and the true factor of safety, the paper is justified.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1627

THE ANCIENT ROMAN AQUEDUCT AT ATHENS*

By James F. Case,† M. Am. Soc. C. E.

Synopsis

As revealed by excavations and examination, the ancient Hadrian Aqueduct at Athens, Greece, is fascinating to the modern engineer. It is about 16 miles long, 30 to 130 ft. deep, and 3.6 by 1.3 ft. in inside dimensions. Although the alignment is crooked, the grade is excellent. Shafts at intervals of about 120 ft. indicate the engineering restrictions of the ancients. However, the materials—brick and mortar—are still in exceptionally good condition. The present (1926) capacity is 2 600 000 gal. per day. No methods of purification were used. The water, obtained from springs, galleries, and wells, evidently has always been hard, forming during the centuries several inches of incrustation. In cross-section this accretion probably shows graphically, by its texture, the sway of Greek power—rapid use at first in the heyday of the State's success, decline to the merest trickle in the mid-period, and renewed intensive flow for the modern city, synonymous with the rise, the fall, and the re-birth of Greece.

Among the duties of the American company that is to finance and construct a new water supply for the Greek cities of Athens and Piræus, is the repair and improvement of the existing aqueduct, which, for nearly eighteen centuries, has supplied the Athenians with water. A public utility of such antiquity, still in service, is a rarity and the writer's opportunity to examine the work and to note the methods of construction, the materials used, and the technical skill displayed, was a privilege not to be disregarded.

* Published in September, 1926, *Proceedings*.
† Cons. Engr., Ulen & Co., New York, N. Y.

The Aqueduct

The Hadrian Aqueduct is named after an illustrious engineer and builder who commenced it in 134 A. D., during his term as Governor of Greece, before he became Roman Emperor. It was completed by his adopted son and successor, Antonius Pius, in 140 A. D. It consists of more than 25 km. (15.5 miles) of masonry-lined tunnels at depths of from 10 to 40 m. (30 to 130 ft.) below the level of the ground. At approximately 10 km. (6 miles) from the city, it bifurcates, one branch leading eastward to the slopes of Mount Pentelicon and the other, northward to those of Mount Parnes.

The water is obtained from springs and subterranean flows, developed by means of galleries, wells, and drains. It is evident that every available water-bearing stratum was seized and utilized, since 465 of such collectors, wells, and branches of the aqueduct have been found, cleaned, and are now in service. The Attic Plain has always been dry and semi-arid. The average annual rainfall is less than 50 cm. (about 20 in.) and the rains are generally torrential, with long dry periods intervening. The intensive development of all underground sources is therefore easily understood.

The tunnel itself was difficult to drive, for the ancients lacked the modern equipment of pumps, forced ventilation, and explosives. Many points of attack were essential. Shafts, therefore, were driven at intervals of from 33 to 38 m. (108 to 125 ft.), and as their depth was relatively great, the located line followed the lowest contours afforded by the terrain, although the length of the line was thereby greatly increased.

The excavated section was small, evidently determined by the space necessary for one man alone to work at each heading. In general, it was approximately 1.70 m. (5.6 ft.) high by 0.70 m. (2.3 ft.) wide, although it varied somewhat with the class of materials encountered. De-watering must have been accomplished by hoisted buckets as no chambers are found where Archimedean screws—the sole applicable method of lifting water known to the ancients—could have been installed.

Much of the excavation was in rock, involving slow and laborious drilling and wedging for its removal. Sections in shale and conglomerate were probably not troublesome, but soft ground in such restricted quarters must have given the engineers and workmen many anxious moments. Slave labor was used throughout, for Rome, as mistress of the world, could supply an unlimited quantity, and the Athenian freeman would do no manual labor.

Materials

The aqueduct is lined throughout with masonry—stone in the rock sections and brick elsewhere. It is evident that the engineers used the nearest available materials. The mortar was the well-known Roman cement, of puzzolana and lime; this was used not only for the masonry bond, but also as a plaster finish on the water-bearing surfaces. Its strength and durability are remarkable, for usually this masonry breaks through the material and not through the joint. The brick are of excellent quality, well burned, and with a clear "ring" when struck with a hammer. They are fully equal to

the modern common facing brick although not so dense as the machine product, as they were all made by hand. Each brick, as it was moulded, was "scribed" by its maker with his thumb and four fingers, thus corrugating the flat sides, probably to give a better bond or possibly as a trade-mark.

The finished section of the brick-lined aqueduct is approximately 1.10 m. (3.6 ft.) high and 0.40 m. (1.3 ft.) wide. In good ground the arch was formed by two flat brick of special size, set on edge at an angle to span the opening and "hipped" at the center bearing as shown in Fig. 1 (*b*). In bad ground the arch was formed of curved brick, approximately 4 by 18 by 30 cm. (1.6 by 7.1 by 11.8 in.) in dimensions, laid on end, haunched at the walls, and joined at the centers (Fig. 1 (*c*)).

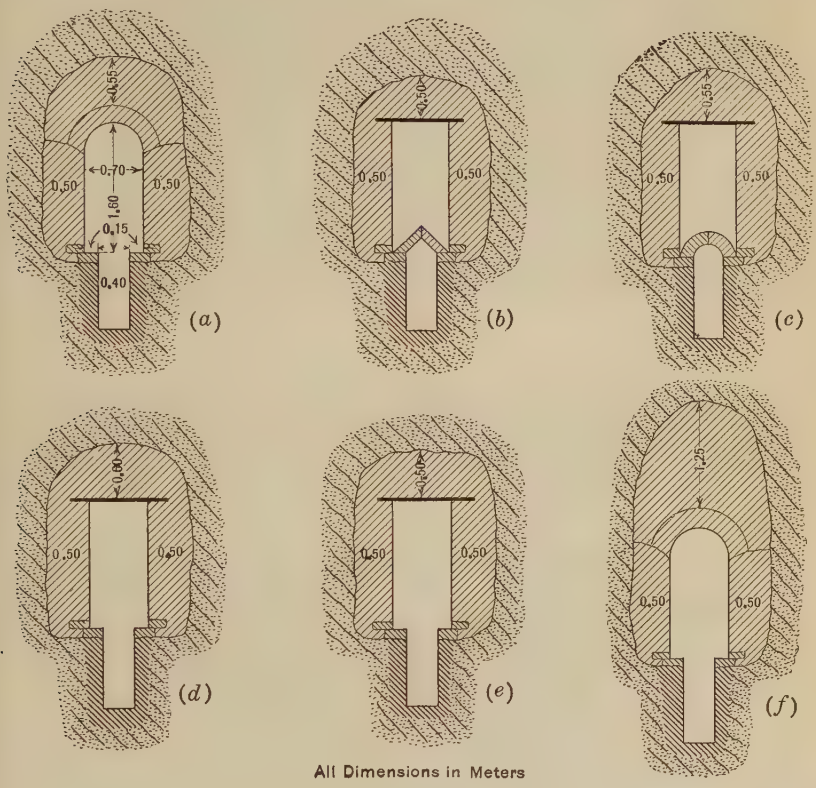

FIG. 1.—VARIOUS SECTIONS OF MODERN LINED GALLERIES USED FOR REPAIR OF OLD AQUEDUCT.

In Fig. 2 is shown a portion of the aqueduct laid bare during the repairs, with the curved arch top exposed. Most of this work is still in excellent condition, although the various "breaks" have been largely occasioned by the failure of the arch when falls from the roof of the excavation have crushed the brick work. This may have been because of carelessness in back-filling, but more probably because of cavities formed by the leaching out of the overlying earth.

Technical Knowledge

The very durability of this work is proof of its excellence. As builders, the Romans rank with the best of all times. Technically, they lacked in knowledge of underlying principles. Not for them was it to know the relation of wetted perimeter and hydraulic radius to flow, although they must have observed the effect of frictional losses, since they sought to make surfaces smooth with troweled plaster. At first glance, it might be thought that the section adopted was due to the space required for one-man headings, but this theory will not hold because the same section was used in the above-ground masonry aqueducts. The fact that they used curved bricks for the arches is sufficient proof that they could have used them for the inverts had they known how to design the most efficient and economical section. More probable is it that they sought merely a hole through which water would flow and discovered that a deep and narrow channel would flush more easily than a shallow one.

The wells and collectors are well built and efficient. The Romans used cut-off walls, or else followed along the water-bearing strata until they found an impervious bedding plane. Thus, they minimized the seepage loss.

Shafts were sunk vertically and lined with masonry when necessary. The levels were easily and correctly run, by means of a right-angled triangle and a plumb line. The grade, or slope, adopted was generally 2 in 1 000, and a modern Wye-level could hardly give better results. The alignment was a different matter. The use of plumb-bobs on either side of the shafts, with visual sighting both above and below ground, was the only method known. The swaying of a line 30 to 40 m. (100 to 130 ft.) long, the pull of a spider's web, or the inaccuracies of a workman's eye over a sight of 5 ft. or less made errors inevitable, and these were multiplied in projection. The explanation first given for the crooked alignment, that the workmen were following water-bearing veins, was disproved when the worst line was found to be in the long rock sections that contained not a trace of moisture. It is clear that each heading must have required frequent corrections in its line, and that as adjoining headings approached each other they were guided by audible signals in making connections. This is also proved by the offsets or wider sections of excavation at the actual junctions. The headings, however, did meet; the aqueduct was finished. It functioned and endured, although its builders lacked the aid of telescopes and cross-hairs. Why criticize the variations from the true tangent?

The worst feature of the small section adopted, was the difficulty—the practical impossibility—of making repairs. With the water running, the aqueduct could not be entered. When the arch was crushed there was no way to reach the break except by driving a new shaft from the surface or a drift from the nearest existing down-stream shaft. The extensive repairs now (1926) in progress include the construction of a lined gallery over the aqueduct and the removal of the arch ring. (Figs. 1, 2, and 3.)

Storage and Distribution

The aqueduct ends at a reservoir on the slopes of Lycabettus Hill high above the central city. This original reservoir was only of 500 cu. m. (130 000

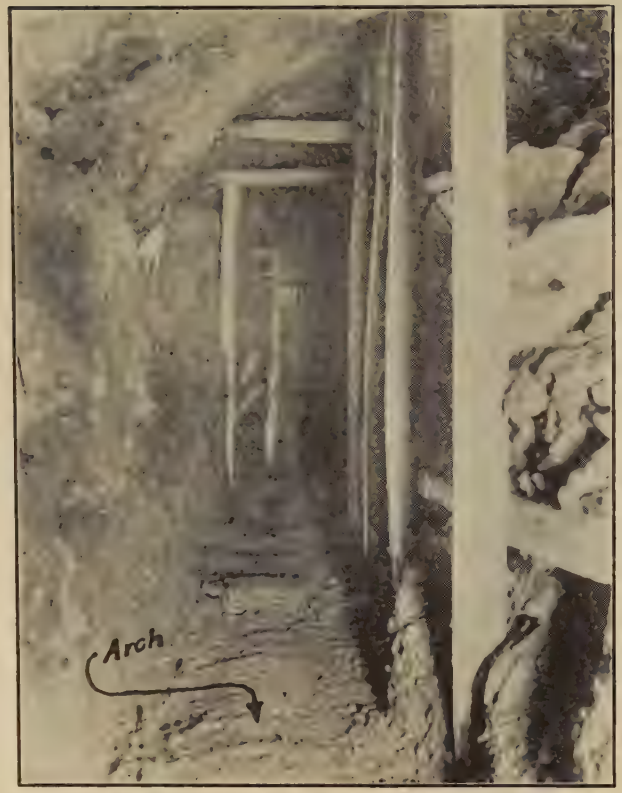

FIG. 2.—EXCAVATION FOR GALLERY OVER AQUEDUCT, SHOWING TOP OF MASONRY ARCH.

FIG. 3.—TIMBERING FOR GALLERY OVER AQUEDUCT, SHOWING TOP OF AQUEDUCT REMOVED.

FIG. 4.—SECTIONS OF PIPE, HADRIAN AQUEDUCT, SHOWING INCRUSTATIONS.

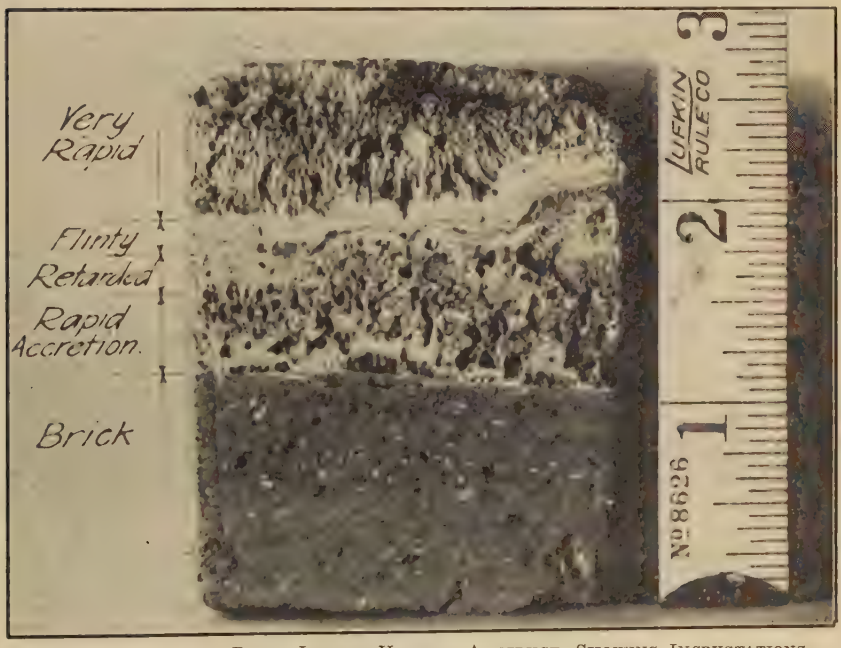

FIG. 5.—SECTION OF BRICK LINING, HADRIAN AQUEDUCT, SHOWING INCRUSTATIONS. SCALE IS IN INCHES.

gal.) capacity, but the city has since built an additional one of somewhat larger dimensions. From thence the water was distributed through conduits borne on masonry arches, one and two stories in height above the streets, with frequent drops to the house level through lead pipes serving the public fountains and public and private buildings.

The pipes were made of lead sheets, shaped to circular form with the ends overlapping, doubled over, and hammered to tightness. Pipes of this kind have been found up to 18 cm. (7 in.) in diameter. The Romans seem to have been ignorant of cast pipes although they were familiar with molten metal, both lead and bronze, and frequently cast figures and utensils.

Quality and Purification

Nor did they know much of water purification. Many of the wells were open to surface contamination. Some of the streams, fed by rainfall, were diverted when in flood, to the aqueduct. The average bacterial count per cubic centimeter to-day is about 5 000. Frequently, it rises to 100 000; not rarely to 1 000 000 and, at times, it is uncountable. It could scarcely have been better in ancient times, but no provision for filtration was made and the only purification was the slight sedimentation in the small reservoir.

Furthermore, the water was hard. Nowadays, it carries upward of 37 parts per hundred thousand of lime. Fig. 4 shows the incrustations in pipes that have been installed only since 1878. Fig. 5 is from a photograph of a section cut from the lining of the aqueduct itself. The incrustation is more than 4 cm. (1.6 in.) in thickness. Using $n = 0.15$ in Kutter's formula for new and clean brick work, the original capacity of the aqueduct was approximately 250 liters (7 cu. ft.) per sec. The incrustation not only reduced the section by fully 20%, but also decreased its capacity greatly through increased frictional loss.

Population Served

No census figures have come down from early times. Various methods, too long to be detailed, have been used to arrive at the probable number of inhabitants. The results indicate that the maximum could scarcely have exceeded 500 000 at any time and of these nearly four-fifths were slaves. Accepting this estimate therefore and considering the capacity of the aqueduct when new, the supply approximated 40 liters (10 gal.) per capita per day. As the supply decreased the population decreased also. In 1835 Athens contained only 5 000 inhabitants. To-day, with a population of more than 600 000 the aqueduct supplies only 10 000 000 liters (2 500 000 gal.) per day, or less than 16 liters (4 gal.) per capita.

A Theory

The writer advances the theory that the incrustation on the aqueduct (Fig. 5) is a geologic record of Athenian history during the last 1800 years. The aqueduct was completed in 140 A. D., at the zenith of Roman power. Put into full service, the deposit of lime grew rapidly, the accretions being noticeably porous and reaching a thickness of 1 cm. (0.4 in.). Rome fell;

the Barbarians swept through Greece and the aqueduct fell into disrepair. Note the gradual increase in density of the deposit. Finally, for centuries its very existence was forgotten, although a trickling stream still oozed through and fell into the reservoir. The deposit then became flinty, susceptible to high polish, and evidently slow of formation. Then came the Nineteenth Century with freedom for the Greeks and the establishment of their nation. The aqueduct was re-discovered, cleaned, repaired, and connected to new wells driven into underground basins in cavernous limestone, the waters of which were charged with calcareous salts in solution. Note the rapid accretions, so honeycombed and porous that they can be flaked off with the finger-nail.

Whether geologic history or not, the theory conforms to the cycle of power, the fall and the rise of Athens in world importance.

Acknowledgments

The writer has consulted many authorities on the subject of this aqueduct and is indebted to the writings of Pausanias, Judeich, Ziller, and Leake, as well as to Messrs. Gounaris, Gorgalas, and Yenidunias, well-known engineers of Athens, who have given him invaluable assistance.

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Paper No. 1628

SAG-TENSION CURVES*

By Edwin S. Parker,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. Marshall Page and J. C. Stevens.

Synopsis

The purpose of this paper is to present a new method of analysis for the tensions and sags in a span of wire or cable, as simple as the common parabola method and as accurate as the catenary method. This will be prefaced by a general discussion of sag tension.

Sag-tension curves are for the use of the man on the job who strings transmission wires. If it is summer, he must not pull them so tight that they will break in winter when they are loaded with ice and are shortened under the cold. Conversely, if it is winter, he must pull them tight enough so that they will not sag below the clearance line on a hot summer day. He must know, therefore, with reasonable accuracy the proper tension for any condition of weather for that particular span and climate. These curves give him that information.

The curve taken by a suspended wire approximates the common catenary, but the analysis of this curve is so complex that the labor involved is not justified when the conditions of loading and strain are so uncertain. Definite loadings of ice and wind are assumed, and the stress on the wire is calculated to its elastic limit, but Nature has a way of doing the unexpected and the actual loading may be quite different from that assumed. In some cases definite physical properties of the wire are known and in others they are merely assumed and may vary considerably; the physical properties (area,

* Published in September, 1926, *Proceedings*.
† Structural Engr., Boston, Mass.

elasticity, coefficient of expansion, elastic limit) will never be exactly the same for any two pieces of wire, however carefully made; on a sunny day the temperature of the wire may be different from that of the thermometer, due to a difference in absorption and reflection of the sun's heat; and, finally, the wires can seldom be strung on an absolutely still day, so that any wind on the wires will give a load that is impossible of estimate by the construction man. For these reasons, any method of calculation that is reasonably accurate and has the same degree of precision for deep curves as for flat ones, is sufficient for the purpose of design.

The methods in use at present for the solution of the catenary are somewhat elaborate and somewhat blind. They require special charts, tables, or diagrams, and the solutions obtained are not obvious, for example, superficial checking does not necessarily prove them to be correct and consistent, as is so easily done by the parabola method.

This parabola method, on the other hand, is simple and clear, so that errors can be readily seen and results clearly understood. It involves direct mathematical principles where the other methods require charts or tables. The parabola, however, has an error that increases as the depth of the curve increases, and therefore applies only to shallow curves, usually of short span. The new method herein proposed is a simple modification of the parabola method. It has all the advantages of clearness, simplicity, and directness, and is as accurate as the more elaborate catenary method.

The solution of the parabola will be first explained in detail, and the new method, the reduction to the catenary, then developed, after which an analysis of unsymmetrical spans will be given.

In the treatment of the problems involved, the following symbols will be used:

Let T_s = tension at support.
T_c = " at center.
W = weight per foot of wire.
S = sag.
L = span.
L_0 = length of wire.
Y = ordinate from catenary to its directrix.

Factor length $= \dfrac{L + L_0}{2}$.

The units are in feet and pounds.

Parabola Method

For illustration, a span of 2 000 ft. has been assumed, with a 0000, 7-strand "Copperweld" cable. The constants are given by the manufacturers and are presumably accurate enough for design. The allowable tension is 7 700 lb., which is one-half the ultimate tension, or the elastic limit. The coefficient of expansion is 0.0000072, and the modulus of elasticity times the area is 2 912 000.

Three loadings are taken in this case: (*a*) Bare wire, 0.663 lb. per lin. ft.; (*b*) wire plus ½ in. of ice, 1.304 lb. per lin. ft.; and (*c*) wire plus ice plus wind at 8 lb. per sq. ft., 1.662 lb. per lin. ft., this being the resultant of the vertical and horizontal loads. In the parabola the weight of the wire per foot of horizontal projection is assumed to be the same throughout the span. This is not true, because as the wire slants up toward the supports the weight per horizontal foot increases. For moderate spans the error is not great and this method can well be used.

This assumption gives a simple relation between sag and tension which is the one common to the design of beams and trusses, namely:

$$\text{Tension} = \frac{\text{Loading moment}}{\text{Effective depth}}$$

The loading moment equals $\frac{W L^2}{8}$. The effective depth is the sag, S. Therefore, the tension at the center of the span, T_c, is:

$$T_c = \frac{W L^2}{8 S}, \text{ or } S = \frac{W L^2}{8 T_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

From the catenary relation, the tension is given by the product of the weight and the vertical ordinate, Y; and as Y increases from the center toward the supports, the change in tension will be the product of the weight and the sag. Therefore,

$$T_s = T_c + WS \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The length of the wire may be found from the expression:

$$L = L_0 + \frac{8 S^2}{3 L}, \text{ or, conversely, } S = \sqrt{(L - L_0)\frac{3 L}{8}} \quad \ldots\ldots\ldots (3)$$

Now the sag and tension for any condition can be found only by solving a long and complicated formula. It is much simpler, however, to make two sets of graphic equations (curves) and find the answers direct from their intersections. It is a case of "divide and conquer", for each condition by itself is easily solved; furthermore, any irregularity in these curves indicates an error.

Equation (1) determines a series of tensions for various assumed sags, one set for each of the three assumed conditions of loading:

(*a*).—*Bare Wire.*—

$$T_c = \frac{0.663 \times 2\,000^2}{8 S}$$

For S = 90 ft.; T_c = 3 680 lb.
" " = 100 " ; " = 3 315 "
" " = 110 " ; " = 3 010 "

(*b*).—*Wire Plus Ice.*—

$$T_c = \frac{1.304 \times 2\,000^2}{8 S}$$

For S = 100 ft.; T_c = 6 520 lb.
" " = 110 " ; " = 5 920 "
" " = 120 " ; " = 5 420 "

(*c*).—*Wire Plus Ice Plus Wind.*—

$$T_c = \frac{1.662 \times 2\,000^2}{8\,S}$$

For S = 100 ft.; T_c = 8 310 lb.
" " = 110 " ; " = 7 550 "
" " = 120 " ; " = 6 930 "

These three curves are plotted on Fig. 1. It is required that the tension at the supports be 7 700 lb. under conditions of greatest load; therefore, substituting T_c from Equation (2) in Equation (1):

$$S = \frac{W\,L^2}{8\,(T_s - W\,S)}, \text{ or } T_s\,S - W\,S^2 = \frac{W\,L^2}{8}$$

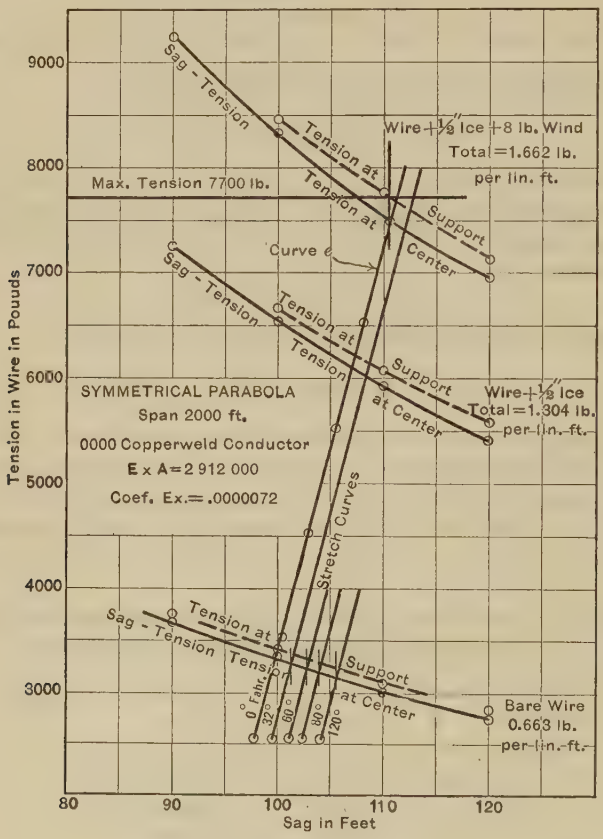

FIG. 1.

Solving by trial, S is found to be 110.5 ft., and from this point the second set of curves or graphic equations starts.

The first set of curves indicates the sag for different tensions under certain assumed conditions of loading. The second set will show the change in sag as the load on the wire changes, for as the wind drops and the ice

breaks off, the tension in the wire decreases, and as the tension decreases the wire shortens. As it shortens, it draws up straighter, decreasing the sag. This decrease in sag, however, produces a slight increase in tension, so that the actual condition is a balance between these two effects.

It is then possible starting with the point where the wire is fully stressed to find its length, and decreasing the tension 1 000 lb. at a time, to find the corresponding series of lengths, and from these lengths, the sags for these several conditions. The resulting curve, (*e*)—the stretch curve—shows the sags that this particular wire will take as its length shortens under a decrease in load. The intersections of this stretch curve for varying loads with the curves of sag tension for the particular loadings chosen, will give the required sags of the wire. The increase in tension as the sag decreases is accounted for in the slopes of the loading curves, for the more the stretch curve bears to the left, the higher up it intersects the load curves. The calculation is as follows:

From Equation (3):

$$\text{Length} = 2\,000 + \frac{8 \times 110.5^2}{3 \times 2\,000} = 2\,016.28 \text{ ft.}$$

$$\text{Elongation for 1 000 lb. change in tension} = \frac{2\,016 \times 1\,000}{2\,912\,000} = 0.69230 \text{ ft.}$$

TABLE 1.—Corresponding Variations in Tension and Sag.

Acting tension, in pounds.....	7 516	6 516	5 516	4 516	3 516	2 516
Difference in tension, in pounds	0	—1 000	—2 000	—3 000	—4 000	—5 000
Elongation, in feet.....	0	— 0.6923	— 1.3846	— 2.0769	— 2.7692	— 3.4655
Length, in feet.....	2 016.280	2 015.588	14.895	14.203	13.511	12.814
Sag, in feet (Equation (3)).....	110.5	108.0	105.5	103.0	100.5	98.2

Curve (*e*) is for 0° Fahr., the lowest temperature assumed, giving the least sag and greatest tension. As the temperature rises, the wire elongates, and this elongation gives it a greater sag. The stretch curves for various temperatures (Fig. 1) will generally be parallel unless the wire is flat in profile.

This analysis has assumed the tension at the center as the stretching force acting on the wire, whereas the mean or average tension will give the real elongation.

The tension at any point of a cable is the tension at the center plus the product of the weight of the wire per foot by the vertical distance from the point to the lowest point of the wire. The average ordinate of the wire above its lowest point will thus be one-third the sag. Hence, the mean tension is the tension at the center plus the product of the weight of the wire per foot by one-third the sag.

On Fig. 1 are plotted the tensions at the supports. The mean tensions will lie one-third the distance between the full and dotted curves. The results found give 1¼ in. less change in sag from using the stretch curve than from using the mean tension; if the tension at the supports had been used as is done

in the catenary methods, the change in sag would have been 2½ in. more than the mean tension. In either case the error is negligible, but the parabola method has one-half the error of the catenary solution.

Parabola-Catenary Reduction Method

As has already been stated, it was assumed for the parabola method (Fig. 2(*b*)) that the weight of the wire per foot of span was the same throughout the span, whereas in the true loading (Fig. 2(*a*)), the weight increases as the wire slopes up toward the supports, and this actual loading will give the true stress in the wire. The catenary method, which takes this into account, is, however, a complicated one. If, however, a length of wire could be found the weight of which applied uniformly over the span would give the true loading moment, and if this length bears a simple relation to the length of the wire and is true for all cases, the true stress in the wire could be found easily. By solving empirically in a long series of trials, this "factor length" (Fig. 2(*c*)) proved to be the mean between the span and the length of the wire.

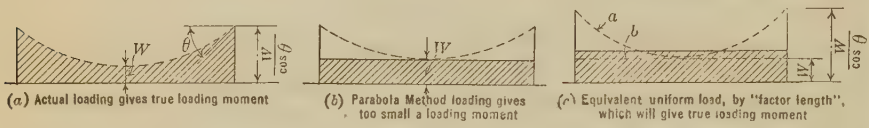

Fig. 2.

Where the true length of wire is taken, by the catenary formula, the results are almost exact.

A complete solution by this method is given on Fig. 3, and this mean length will be called the "factor length".

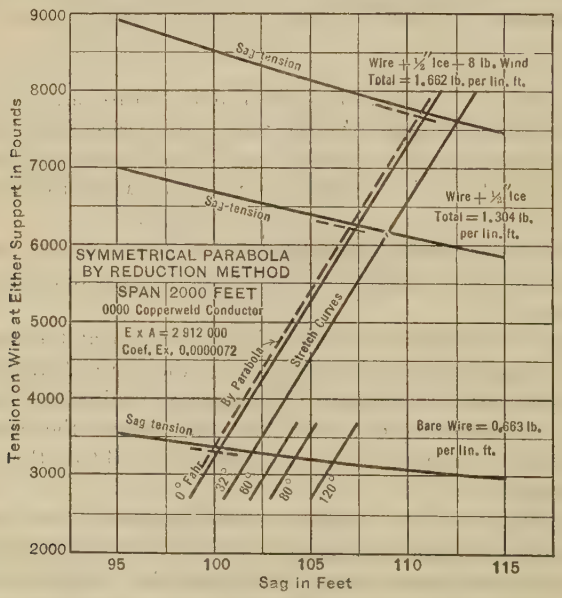

Fig. 3.

Unsymmetrical Curves

Unsymmetrical sag curves present an elusive problem. The usual method is to treat the curve as if it were symmetrical taking the vertical sag from the chord to the tangent parallel to the chord, or, more simply, the vertical distance from the center of the chord to the curve. Even if some allowance is made for the greater length of the chord over the span, this method cannot be accurate because the tension is affected by the total sag, as will be shown.

The solution here presented is reasonably accurate and depends on elementary rather than complex principles, so it has the advantage of clarity. The stress in an unsymmetrical curve is determined by the distance from the low point to either support and not by the clear span, either part being in effect one-half a symmetrical span as far as analyzing goes. As the sag changes the low point moves, so that the equivalent span length becomes a variable quantity. The steps of the actual analysis are as follows:

First.—Find the distance from the low point to the higher support for a range of chosen sags, by writing the equation of the parabola through two points tangent to the X-axis.

Second.—Find the length of wire for this part-span (and at the same time for the shorter part-span).

Third.—Find the sag-tension loads for spans that are twice these longer part-spans, and lay out sag-tension curves.

Fourth.—Make an auxiliary curve of the length of wire for each sag.

Fifth.—Intersect the sag-tension curve by the maximum tension (7 700 lb.) to obtain the starting point.

Sixth.—Project downward to get the length of wire at this sag and tension, and thus proceed graphically to find points on the stretch curve.

This method is accurate but is neither simple nor easy, as the equations present considerable practical difficulty; but here, again, an approximation is possible. From the shape of the parabola the quantity, $\frac{W L^2}{8 T_c}$, will give the perpendicular distance from the center of any chord to the curve, and this will give the data required for stringing by sights.* The only problem then is to get the factor length again and to allow for the change in tension from the higher support to the true low point.

By empirical analysis, the length of the wire is found to be nearly

$$\frac{L_0}{\cos \theta} + \frac{8 S^2}{3 L_0}$$

in which, θ is the angle of slope of the chord. The T_c of the formula is pure horizontal tension, at the low point. The quantity, T_s, is found by allowing for the rise from the low point to the center of the span and for the distance from the high point to the center of the chord. From this the true sag at 7 700 lb. tension is obtained. From this point, the solution is the same as for the symmetrical curve (see Fig. 4).

* In stringing by sights, grade marks are set on the towers, and the bottom of the wire is sighted from tower to tower.

Illustration of Solution for Unsymmetrical Curves

Assume an unsymmetrical span, as in Fig. 5. The rise, r, from the lowest point to the mid-point is approximate. In this example, an r of 23 ft. is used. The properties of the wire are the same as in the previous example.

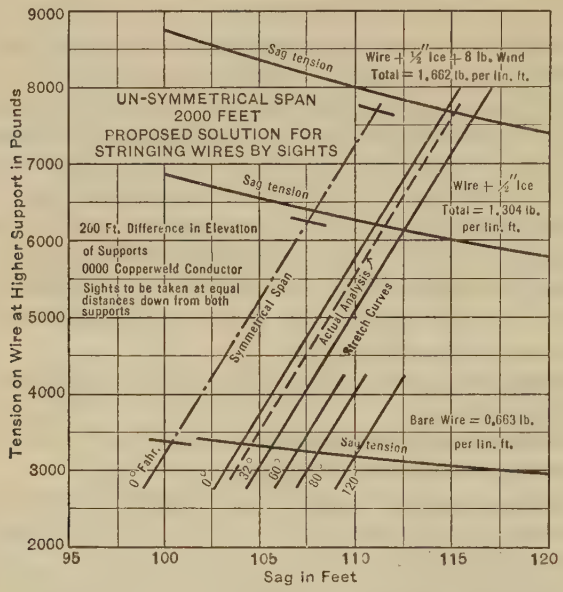

Fig. 4.

The tension at the low point of the wire will be,

$$T_c = T_s - (100\ W + W\ S + W\ r) = \frac{W\ L^2}{8\ S}$$

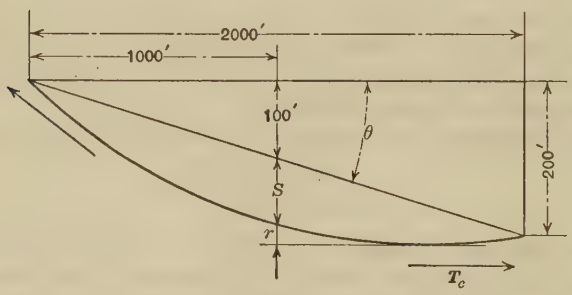

Fig. 5.

whence,

$$S\ (T_s - (100 + r)\ W) - W\ S^2 = \frac{W\ L^2}{8}$$

Solving for S,

$$S\,(7\,700 - 123\,(1.662)) - 1.662\,S^2 = \frac{1.662\,(2\,000)^2}{8}$$

or,

$$S = 114 \text{ ft.}$$

Likewise solving for L,

$$L = \frac{L_0}{\cos\theta} + \frac{8\,S^2}{3\,L_0} = \frac{2\,000}{0.99473} + \frac{8\,(114)^2}{3\,(2\,000)} = 2\,027.97 \text{ ft.}$$

Then the "factor length" is,

$$\frac{2\,000 + 2\,027.97}{2} = 2\,014 \text{ ft.}$$

Using this value, proceed with the solution as in the case of a symmetrical curve, where solving again gives the true value of S from the equation,

$$S\,(7\,700 - 123\,(1.662)) - 1.662\,S^2 = \frac{1.662 \times 2\,014 \times 2\,000}{8}$$

Conclusions

1.—The methods here explained show that for shallow curves the parabolic method of treating sag tension is satisfactory, but that for deep curves the error is great. These errors are shown graphically on Fig. 6, and from this the proper method of analysis can be chosen when the degree of accuracy is determined.

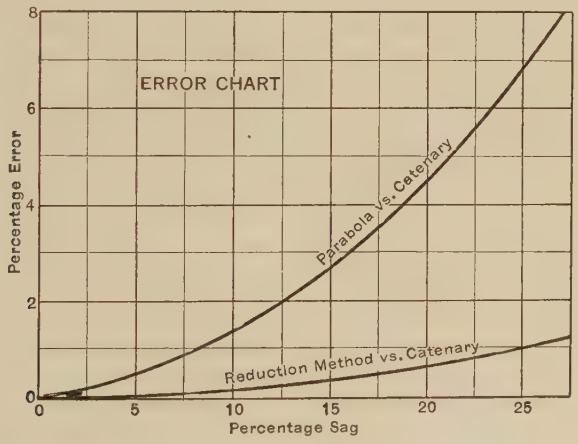

Fig. 6.

2.—If the wires are to be strung by tension* it makes little difference which method is used, as all the values start from the maximum allowable tension and are generally parallel.

* In stringing by tension, a tension meter is used at the end where the wire is being pulled. As this point is usually near the ground, an allowance must be made for the difference in pull due to the elevation lower than the point of maximum stress.

3.—For stringing by sights there is a wide variation in sag unless the unsymmetrical curve is really analyzed. On Fig. 4 this variation is shown for the 0° Fahr. curves by three methods. It is obvious that the analysis of unsymmetrical curves should not be slighted when the wires are to be strung by sights.

4.—If the wires are to be strung by tension, the reading on the meter should be $T_s - W(S—Y)$; if the stringing is done from the lower support of an unsymmetrical span, the reduction in tension at the meter is the weight of wire per foot times the vertical distance from the higher support to the meter.

DISCUSSION

W. Marshall Page,* Esq. (by letter)—Mr. Parker's suggestion of using a "factor length" instead of the usual span length in the parabola formula, and of making provision for the tension at the support rather than using the tension at the low point of the span, is a decided improvement. The increase in accuracy obtained makes this method with his modifications equivalent to the catenary methods.

For short-span construction, the ordinary parabola method has been as satisfactory as the various catenary methods. It was, however, subject to some criticism in the calculation of sags for long spans, as these showed a greater safety than actually existed, or gave a stringing sag less than that obtained by other methods. The error tended to decrease the actual factor of safety. Probably one of the main objections to the ordinary parabola method was the uncertainty regarding this error.

It is true that many of the other factors that enter into a sag problem are also subject to considerable variation, but these have been generally understood or accepted, and have been provided for by using conservative assumptions. The incidental errors were usually in such a direction that the actual factor of safety was increased. For example, cables are often 10% stronger than the rated strength used in the design calculations.

Mr. Parker gives a very clear demonstration of the parabolic method for the solution of sag problems, and cites the advantages of this method. The basic formula is simple and easily handled. The graphic solutions as given in Fig. 1 definitely show the relation between the sag and tension of a suspended wire under various conditions of loading. This is a very desirable feature for those who only occasionally have sag problems.

The use of the "factor length" and the actual tension at the support should be adopted by all who are using the parabola method. It retains all the advantages of the parabola and the increase in accuracy makes it applicable for the calculations of spans of all lengths.

J. C. Stevens,† M. Am. Soc. C. E. (by letter).—The author presents a method both simple and practical for the solution of the sag of cable spans. A number of years ago the writer prepared a diagram for the erection sags to be given cables used in supporting cars for stream-measurement work.‡ The requirements in that work principally hinged about the fact that the operator must be in no danger of touching the water when the stream was in flood.

It is often impracticable to give the cable a predetermined tension, and the writer outlined a method whereby the required tension at erection could be given by an "erection sag," which would be inversely proportional to the tension. It is a simple matter to draw a cable up to a given sag, but not always so easy actually to measure the tension.

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† Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

‡ *Engineering News*, May 6, 1909, p. 483, and July 15, 1909, p. 80.

One difficulty encountered by the writer—and the author has thrown no light on it—was to correct for the effect of temperature. It is simple once the linear coefficient of expansion is known. The writer questions whether the coefficient for the material of which the cable is composed will apply to the cable itself. In reply to the writer's request to the U. S. Bureau of Standards to forward him data on this question, it was stated that the Bureau had no information on the subject.

It appears that the coefficient of linear expansion of a cable as compared with a wire of the same material must vary with the lay of the cable and also with the tension it is under at the time. If the author has any data on this point he will render the profession a service by publishing them.

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Paper No. 1629

THE ENGINEER AND THE TOWN PLAN*

By James Ewing,† Esq.

With Discussion by Messrs. Noulan Cauchon, Morris Knowles, William T. Lyle, Charles H. Paul, and Stephen Child.

It is often asked why the engineer does not bulk more largely in public life and in the public eye, as do members of the other professions. The answer has been given that he has little inclination that way, is too busy with things of much greater importance, and is not given to self-advertising.

All this may be true, yet these answers only give rise to another question, as to whether or not his influence for good in the community and as a shaping, regulating, and determining force, is not greatly lessened by his lack of interest in public affairs.

The engineer by his painstaking concentration on details and methods has to a great extent surrendered his grip on the larger issues and has become the servile instrument, the mere technical tool in the hands of men far beneath him in intelligence, in thorough understanding of some particular problem, and in grasp of its essential features. He occupies a position midway between that of the enterprise promoter and the manual laborer; he is supposed to be the brains of the combination, but is usually allowed less recognition and even less pay than either.

The result has been that in popular estimation he is held in a position fittingly denoted by a former Canadian Cabinet Minister when he said "Engineers should be on Tap but not on Top". Yet the training and education of the engineer cover a greater field, a wider range of scientific study than that of any other profession. He must learn as much as possible of the nature

* Presented at the meeting of the City Planning Division, Montreal, Que., Canada, October 15, 1925.

† Cons. Engr.; Pres., Town Planning Inst. of Canada, Montreal, Que., Canada. Mr. Ewing died on November 5, 1926.

and properties of all kinds of matter under varying conditions. All the physical sciences must be within his ken, their special manifestations and the laws that govern them, and "the great forces of power in Nature" must be captured, tamed, and harnessed by him and "converted, adapted, and applied for the use and convenience of man".

In addition to all this, he must have more than an ordinary acquaintance with business, administrative, and financial dealings. In fact, the engineer is supposed to be so much a living embodiment and epitome of all the sciences as must be beyond the capacity of the most gifted mortal effectually to encompass. The consequent tendency has been to specialize into various branches of expert practice.

Thus, there are railway engineers, tramway engineers, hydraulic, municipal, mechanical, mining, metallurgical, electrical, chemical, bridge, and building engineers; and these, in turn, have become subdivided into experts in steel construction, reinforced concrete, etc. So to-day the brand of engineer that formerly was so all-embracing, so well known, and at the same time so dignified, termed the Civil Engineer, has almost become a thing of the past.

Although specialization may offer undoubted advantages, there is grave danger in over-specialization presenting a detached and only partial side of the case with incomplete and inharmonious results. During the last decade a new species of engineer has achieved some degree of recognition and success, that is, the Town Planning Engineer. Hitherto he has found little favor or esteem in the eyes of his brother professionals, who are inclined to look askance at him as a sort of hybrid individual—a combination of architect, surveyor, and landscape specialist with possibly a certain percentage of engineering knowledge thrown in.

If the truth were known and appreciated, however, it would be realized that town planning is nothing more nor less than fundamental engineering, the solid substructure on which ordinary diversified engineering should be based. Of course, the term, town planning, is a misnomer and is used only for want of a better designation that may prove more apt and at the same time generally acceptable. Although it may have originated with special reference to the city or town, it has assumed a much wider significance, embracing regional planning and even country-wide planning; and who knows if when perfervid nationalities shall have pooled their fancied differences, it may not reach out into world-wide planning.

Canada is an undeveloped land of wonderful natural resources, of vast fertile plains, of abundant forests, and of prolific oil and mineral-bearing regions. Besides this there are immense water powers either partly harnessed or running to waste and except for a few of the larger cities, only a far-flung, meager, and attenuated settlement to develop it. In fact, with a population sufficient for a few counties an attempt is being made to run half a continent, all at once.

Three or four decades ago the idea was conceived that the one thing needful to open the door to all these riches was efficient transportation. So railways were built with a lavish hand and with sublime faith in a beneficent and

overflowing future. Not only were they built where they were needed or gave reasonable promise of success, but they were built broadcast. Moreover one railroad system was not enough; several must be had to break the dreaded monopoly and ensure healthy competition. So, duplication and triplication became the order of the day, with lines interlacing and paralleling each other almost within a stone's throw, and positively cutting each other's throats. And the engineers—brave, brawny, and brainy fellows, inured to hardship and unafraid of danger or of work—drove these lines through wilderness, swamps, and sandy deserts, across wide waterways and rocky barriers, along cliffs and canyons, and over lofty mountain divides, from ocean to ocean. These lines are to-day monuments of engineering enterprise and skill, and, at the same time, lasting memorials of pitiable judgment and radically bad planning.

As long as the job and their health lasted, the engineers drew their meager monthly salaries, while their masters—the contractors and company promoters —became millionaires. To-day, the nation goes staggering under the burden of a colossal debt for capital account, with yearly deficits in operating expenses of many millions of dollars pyramiding one on top of the other.

Politicians are trying to fasten the blame each on the shoulders of the other and succeeding only in demonstrating the elemental absurdity of lawyers, doctors, and self-interested business men, settling on political lines such economic matters of supreme moment without mature deliberation and expert advice.

If the engineers were not consulted, as they should have been, whose fault was it? They did know all about "how" to build railways, but as to the "why" and the "wherefore" of the whole thing and the "whither it might be tending", that would have been new to them. They were past-masters in the science of the most economic location and construction, long and short haul, borrow and overbreak, ruling gradients, velocity grades, vertical curves, and spiral alignment, but what did they know or care regarding the utility or necessity for these railways in relation to possible development, proximity to markets, and existing or prospective industries, and least of all for the accommodation and convenience of the shipping and traveling public. In other words, they were splendid engineers, but altogether indifferent planners.

Nor is it only in the open country where the ill effects of indiscriminate planning is felt. The cities themselves are honeycombed with rival surface lines and terminals, obstructing and paralyzing the street traffic, creating congested conditions, blighted districts, and stunted growth, and cramping and binding the very vitals of the city in their iron strangle-hold.

Take the question of grade crossings, for instance. It formerly was considered good practice and rather clever engineering if these could be put over on an unawakened public wherever possible. The almost inevitable toll in loss of life and limb was something to be lamented, but it was up to the victims themselves to avoid it.

More than a dozen years ago a branch line was projected starting from the main line of a great railway company in Montreal and extending through

an adjoining rapidly growing municipality. The plans were prepared and sanctioned by the Board of Railway Commissioners for Canada. For the city portion, it was proposed to cross the streets by overhead bridges, but as soon as the city limits were reached, it was designed to descend to the ground, involving a series of grade crossings. As construction advanced the people of the district affected realized what was happening and strenuously objected, but to no avail. They then determined to take the law into their own hands, and repeatedly, during the night, demolished the work that had been done the previous day until the projected construction was discontinued pending an appeal to the Courts. That municipality has since been incorporated within the city and is already one of its busiest and most populous sections. The projected line was never finished, and later experience has shown that the comparatively ignorant men of that municipality were right, and the railway company and its engineers, together with the Board of Railway Commissioners, were entirely in the wrong.

Another important question is that of the location of bridges. One would think that utility, convenience, and directness in relation to the origin, volume, trend, and destination of existing and expected traffic would be the primary and ruling considerations in the location. Too often, however, these questions are not studied, the determining factors being initial cost of right of way and construction, stability in foundation, length of span, etc.

This may be excellent engineering but it can scarcely be considered good planning, or even good business, if, in a year or two, a comparatively trivial saving in first cost is to be offset by the perpetual cost of unnecessary extra haulage, piling up in the course of a number of years to an astonishing aggregate sufficient to overbalance the entire capital cost of the undertaking.

The question, "Why is a street?" is one which might profitably engage the attention of the engineer. Is it merely for access to the houses and buildings thereon or to be used as an arterial thoroughfare for through traffic? It seems axiomatic that a roadway should be proportioned and paved according to the amount and nature of the service it will be expected to render.

An engineer called on to design water distribution or sewage disposal services would scorn the absurdity of using pipes and conduits all of about the same dimension, yet that is exactly what is done with streets. Although the engineer may not always be culpable in the first place, by his mute, unquestioning acceptance, he is accessory and contributory to the perpetuating of the wrong and in rendering remedial measures impossible.

It is amusing to observe the trouble and care that an engineer will take in the chemical constituents and handling of his paving material to save a few cents per yard in its cost, and straightway set to work and plaster it all over a street of useless and extravagant width.

The City of Montreal, with its unrivaled strategic situation in the heart of a continent and at the head of ocean navigation, has been singularly endowed by Nature, being bounded on all sides by broad flowing rivers, with gentle slopes rising to a commanding mountain in the midst of the city. This mountain is of course the pride and glory of the populace and is an inestimable

rest and recreation ground. On either side of the mountain east and west extends an escarpment for the greater part of the island.

If the city had been planned intelligently, systematically, and with foresight, to suit the topography, it would have been ideal in every respect. It was allowed, however, to grow simply by accretion on more or less rule-of-thumb rectangular lines.

The streets as a rule run either parallel to the river or straight up the face of the hill. The cost of these steep grades to the people of Montreal during the last hundred years would probably be enough to buy all the land in the city several times over. Yet no serious or determined effort has been made to deal with this difficulty except perhaps in one instance where, some years ago, it was decided to reduce the grade on a certain street by the primitive engineering method of cut and fill. As the work progressed it was found that at one part the grade of the roadway was level with the chimney tops while at another the houses were perched on a hill more than 20 ft. high. The city was deluged with threats of lawsuits amounting to more than $1 000 000, and it was found that the cheapest way out of the difficulty was to undo the work that had been started and put the street back in its original condition. Whether this attempted operation was bad engineering it might be unwise to assert, but certainly it was not according to accepted city planning methods which would have suggested a less expensive and more satisfactory solution of the trouble.

Although Mount Royal is a wonderful boon to the people of Montreal, it can scarcely be considered as such where the question of traffic is concerned, for all the eastbound and westbound traffic is compressed and forced through a narrow bottle-neck with results that are daily becoming more intolerable. To aggravate this, what may be considered the main thoroughfares are entirely too narrow for the work that is expected of them, and in addition the longest sides of the rectangular street blocks have been set crosswise to the main flow of traffic, thus providing fewer channels and creating more stringency.

Altogether Montreal has a traffic problem of first-class magnitude, the terrible incessant and ever-increasing expense of which, although it may not appear on the ledger, is none the less real. Down-town property values especially are deteriorating and business centers are shifting not by natural extension but by tearing away from their moorings and leaving dilapidation and decay behind, while new high buildings are going up elsewhere, effectually blocking even the possibility of improvement. Hitherto no solution has been found for the trouble except the piecemeal, blind, and arbitrary widening of highly developed main thoroughfares, at an aggregate cost of more than $20 000 000, with altogether incommensurate results, and with the effect that, owing to the dislocation and turmoil of the operation, high-class business, that capricious jade, has flitted off to other regions and refuses to return to her old-time haunts.

On account of the small reserve left in the municipal treasury, recent efforts have been confined to frenzied and practically futile endeavors to ease the situation by traffic direction, regulation, and restriction, limitation of parking privileges, one-way streets, etc. This sort of thing is nearing the

end of its effectiveness, and the question arises as to what can be done next. It has been suggested by tramway interests that as they are the carriers of the largest number of passengers, they should have paramount rights, and that at least during rush hours motor traffic should be ruled off the highways and relegated to the byways. On the other hand it has been contended by the motor people that the tramcars, by their bulk, unwieldiness, and rigidity to the track, together with their many stops and switching operations, are themselves the greatest sinners, and consequently they should be removed from the surface of the streets.

Strange to say, whatever quarrel may exist between these two interests they are at one in their contemptuous and overbearing, "get-off-the-earth" attitude toward the luckless and defenseless pedestrian, the most multitudinous of them all.

The foregoing may be taken as incidental examples of the general trend of things in this mad rush of modern activity and enterprise, where we seem to be able to see and to grasp only one side of one thing at a time; where there is such a lack of calm and careful deliberation and of thorough and conscientious study of cause and effect; where we fail to appreciate the position and point of view of the other fellow, and the spirit of co-ordination and compromise is conspicuous by its absence. Is it any wonder, therefore, if in such a process there should be so much misspent, thwarted, and dissipated effort, with disappointing, confusing, and chaotic results?

Perhaps amid all these clashing, warring, and grinding elements comprehensive town and country planning is the one clear star shining through the murky atmosphere, ensuring or striving to ensure for every interest and every individual the nearest approach to a square deal, a fair show, and a decent and fitting place in the sun.

DISCUSSION

NOULAN CAUCHON,* ESQ.—The speaker appreciates the lively interest that the American engineer has taken in town planning. Canadians have not, so far, been fortunate enough to interest the Engineering Profession as a whole in it. It is hoped that these discussions will assist in opening the eyes of Canadians to what Americans have recognized, that is, the future of town planning. The ideas of the Town Planning Institute are considered very idealistic. Not that it fails in the practical side, but it has to educate the public to the need of it. Engineers have lacked public recognition where they have not given public service. They should give some part of their time in service to the public.

The Town Planning Institute was founded in Canada on the principle that all men coming to it have to be recognized by their own professions before it admits them. In other words, it is sort of a "Post-Graduate Institute" and is based on the principle that there be no overlap of function. It wants the architect to know enough about engineering not to put a building where he can see that a street will have to be widened, and for an engineer to know sufficient of architecture and art to restrain him from mutilating beauty, existing and potential. It desires to have an overlap of knowledge, but not an overlap of function.

Town planning should be based on a biological aspect. A city in which the streets are narrow and dark does not provide a biological condition fit for the maintenance of life. It is simply a problem of translating that view into engineering terms. The engineer who is planning the city is a biologist and has to know and plan for the conditions under which life can exist and thrive. It is not new as knowledge, but is simply a focusing of science to the purpose of human betterment—in other words, human efficiency. The congestion of streets is simply a cancer in the organism of the city.

Business men have said, "Why do you not let the city grow naturally"? It cannot grow naturally because it is a machine in which to live. What will happen unless the engineer designs the city so that it will not go to pieces? A city expands by accretion merely, being devoid of inherent biological balance and checks.

The question of highways is an important issue. Canada is over-capitalized in railroads. The United States has the same problem, but it can carry it better because it has a larger population. Every one has gone crazy on the subject of highways. Let us not run away with the idea that we can build an unlimited number of highways, or an over-capitalization in highways, as in railways, will result. Instead, as an engineering policy, the effort should be toward intensive cultivation and population along those highways to sustain their cost. It is not a political question but an economic one. As it is, the cities are carrying the burden. This is simply a thought for engineers.

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Another important thing in respect to highways is to obtain and preserve ruling grades on them. The first difficulty in a grade separation is that the railway wants to retain its ruling grade and will give good reasons for so doing. The point is that both the railway and the highway should have and preserve their ruling grades and divide the cost.

MORRIS KNOWLES,* M. AM. SOC. C. E.—The author has been frank, with usual English bruskness, in telling engineers of their faults and failings. Much of it is truth they need to know. He is not less veracious than he is forceful in his reference to the "brave, brawny and brainy fellows", who are wonders in producing "monuments of engineering enterprise and skill". These may be "lasting memorials of pitiful judgment and radically bad planning"; and—as he says again—the work of "splendid engineers but altogether indifferent planners".

Another discussor suggests reasons why other professions succeed in having public influence. Probably he did not intend to imply, and most engineers will not agree, that in order to have influence it is necessary to be servile. Honest, efficient service may be performed without fawning. The reason for lack of influence lies deeper. The engineer is not so gregarious as many other types of mankind. He is accustomed to deal with material things more often than with human beings. While he frequently works in groups, his labors and discussions are objective rather than subjective; they deal with the stress of concrete beams rather than any stress of public and human affairs.

This is somewhat different from the other professions. The clergyman, the lawyer, and the doctor frequently gather together and discuss public questions quite as often as professional matters. They sometimes settle important public affairs without having definite knowledge. The engineer is prone to discuss the technicalities of a thing—its detail rather than its conception and planning and its effect on other public activities. In so far as engineers have this lessened public contact with each other and with other groups, and participate less in the discussion of political and public questions, their influence is less than that of other professions.

Engineers will agree that the basis of city planning is engineering, but certainly they can have more influence in the community if they can "put over this idea" to others—can cause them to realize that the fundamental backbone of any planning, before the design or execution, is the acquirement of information of fundamental engineering nature, such as topography, distances, elevations, mapping of natural features, etc. Of course, the acquirement of this information is not designing, but one must plan a place for people to live, a location for stores in which to do business, and industries in which to work, with suitable surroundings and facilities for each. Adequate highways of sufficient width for convenience and transportation and the relative placing of workshops, stores, and homes in suitable locations, without jarring juxtaposition on each other, is primarily an engineering study. He who has been doing this from time immemorial is the city engineer. True, some of it has been badly done but much of it has been done so well as to make water systems, sewerage systems, highways and other forms of public utilities planned

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for the future rather than the immediate present. Thus it may be said without fear of contradiction that the original city planner was the early and original city engineer.

Surely it will be worth while to remember this, to preach this; yes, but to execute better. At the same time engineers have much to learn from the kindred professions in the line of conception, in design, in harmony, and in execution, and likewise much to learn as to the advantage of public discussion, preachment of their ideas, and selling of their services. They may then extend their influence and make their contribution more worth while. In other words, engineers may be of service without being servile.

William T. Lyle,* M. Am. Soc. C. E. (by letter).—It is to be regretted that certain papers published by the Society are read only by those attracted by the titles. The paper by Mr. Ewing will interest all engineers engaged in city planning; but by reason of breadth of vision, keen criticism, and constructive wisdom it deserves to be read also by all other members.

The Civil Engineering Profession during the last century has passed through a cycle of development. As differentiated from Military Engineering it concerned itself with civil pursuits. The earlier civil engineers were engaged in the projection, design, and construction of railroads, canals, power plants, harbors, bridges, and many other public works. Later, came the specialized subdivisions mentioned in the paper, some at first little higher than craftsmanship. The lowly origin of Sanitary Engineering is within the memory of many engineers still living, and articulated bridge building is known to have had its beginnings in carpentry. An examination of the files of the technical press forty years ago is both instructive and amusing.

To-day, the drift is toward higher standards and greater usefulness and, strange as it may seem, the city planning and regional planning engineer is pointing the way. Engineers must be more than mere technicians; they cannot in self-respect be content to follow without question the direction of those who employ them; they must exercise a projecting and guiding control in every plan. When they have thus asserted themselves in determining the feasibility of a project as well as the design of the works, they rise to the level and dignity of what might be called planning engineers, for want of a better designation; in other words they become engineers in the highest possible sense.

Thus, although highly specialized subdivisions will always be necessary, a strong current is running in an opposite direction, that is, toward unification. Engineers of broader vision and enlarged power are indispensable to the great works of modern civilization—works which are intimately related to many others and which cannot be properly planned without a comprehensive and far-seeing vision.

Charles H. Paul,† M. Am. Soc. C. E.—In the work of city planning, the engineer is given an opportunity to get in close touch with the public, such

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† Managing-Director, Dayton Industrial Assoc.; Cons. Engr., Dayton, Ohio.

as is not offered in many other branches of engineering. The speaker has had the privilege of serving on the City Plan Board in Dayton, Ohio, and that work has resulted in the fullest co-operation between the City Plan Board, the Real Estate Board, the Chamber of Commerce, and various other business men's organizations. Recognition of the need of engineering service on the Plan Board was shown by the fact that two of the seven members appointed were engineers, and one other was a plant executive with a definitely engineering turn of mind. There is no question that these city planning studies are bringing to the minds of the public the value of engineering training and engineering analysis in the solution of many of the problems which are faced by the municipality.

One of the problems solved by this Plan Board was a serious grade-crossing situation. The elimination of grade crossings in Dayton had been discussed for fifteen years. Various solutions had been offered, but the situation had never been studied in a manner that would give full consideration and weight to the advantages and disadvantages of the various possible solutions. The City Plan Board had the problem analyzed in that way, and had taken into account, and made a part of the same problem, the re-arrangement of streets and thoroughfares in the vicinity of the tracks, the freight stations, and the passenger station. Although the cost of grade-crossing elimination is bound to increase year by year as the actual work is postponed, it has been remarked publicly by one of the city officials that the delay and increased cost, in this case, were well worth while because the present solution is so much better than any that had been offered before.

The city planning engineer, just by the nature of his work, is dealing directly with the public most of the time. He has the opportunity to broaden and develop himself because of this direct contact with the various public groups, and, at the same time, he has the chance to show the public, both as individuals and as groups, that engineering training and the engineering approach to a municipal problem usually results in the most direct, satisfactory, and economical solution. Contributions of this kind by engineers to civic work will do more than almost anything else to put the profession in its proper place in the eyes of the public.

Stephen Child,* M. Am. Soc. C. E. (by letter).—The author makes the assertion that "town planning is nothing more nor less than fundamental engineering, the solid substructure on which ordinary diversified engineering should be based"; and, after alluding to certain phases of the great problem, namely, transportation, grade crossings, bridges, streets, traffic, etc., and the serious conflicts that have in some instances accompanied misdirected effort to solve some of these, closes as follows:

"Perhaps amid all these clashing, warring, and grinding elements comprehensive town and country planning is the one clear star shining through the murky atmosphere, ensuring or striving to ensure for every interest and every individual the nearest approach to a square deal, a fair show, and a decent and fitting place in the sun."

* Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

The writer who has studied and worked on many of the phases of this great problem for more than thirty years, believes this last statement to be true, but takes issue with the first one as being too narrow a view of the problem. There is indeed need for the "solid substructure" of exact engineering science, but there is need, too, for much more than "ordinary diversified engineering", for vision, for imagination, and for a knowledge of the principles of design.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1630

AERIAL SURVEYS FOR CITY PLANNING*

By Gerard H. Matthes,† M. Am. Soc. C. E.

With Discussion by Messrs. Arthur S. Tuttle, F. L. Olmsted, Charles N. Green, Theron M. Ripley, and Gerard H. Matthes.

In approaching this subject it is desirable first to consider what kinds of maps are best suited for city planning and zoning, and, next, in what way aerial photography may be used to advantage in producing such maps.

Maps Versus Plans

There appears to be some confusion regarding the nature and functions of a map and regarding those of a plan or working plan. It is common to find municipal authorities and their engineers expressing the desire for a map of their city produced by aerial photography that will also serve the purposes of a plan on which to lay out construction work and measure contract quantities. To clear up any misunderstanding on this score it may be well to state at the outset that a map, whether produced by aerial or by ground survey, as a rule, cannot be used as a working plan. A map serves to visualize the broader aspects of city problems, including their major details, and is, therefore, especially suited for planning purposes, whereas a plan is a large-scale drawing needed for making construction layouts that usually show no details except those required for a specific project.

Maps for City Planning

A map suitable for city planning may be defined as one showing comprehensively and in fair detail those features that are of prime interest in considerations affecting traffic, transportation, population, disease, crime, acci-

* Presented at the meeting of the City Planning Division, Montreal, Que., Canada, October 15, 1925.

† Cons. Engr., New York, N. Y.

dents, education, recreation, building uses, water supply, and sewerage—both as regards existing conditions and future requirements. The scale is best chosen so as to keep the city area within the scope of one sheet of paper, or, if the area is very large, on the fewest possible number of sheets, otherwise no comprehensive view is afforded. No high degree of precision is required but the relative positions of objects shown should be sufficiently accurate to permit of determining approximate distances and grades for planning the location of thoroughfares, sewers, water mains, parks, parkways, and rapid transit lines. Scales ranging from 200 to 600 ft. to 1 in. and contour intervals of 5 or 10 ft. appear to be best suited to city planning purposes. Much will depend on local conditions and the size of the budget as to what particular scale and contour interval should be selected in any given case, and no rule could safely be promulgated that would give uniformly good results in all cities.

It is a lamentable fact that few towns and cities in North America possess maps answering the requirements of the city planner. The plans or plats found in the average city engineer's office, showing sewer or water main locations, are usually on scales as large as 40 or 50 ft. to 1 in., and if at all complete consist of a great many sheets that afford no comprehensive oversight such as is paramount for intelligent "planning". Often a great deal of miscellaneous survey data are available in the city engineer's files but in the absence of control or co-ordinate systems it cannot be utilized for constructing a map. In the majority of cases lack of funds and failure on the part of city authorities to recognize the importance and value of accurate maps appear to be the principal obstacles. As a result, most city maps are not much more than mere skeleton outlines of the street plan with no pretense of accuracy or of showing buildings and other important detail.

Brookline, Mass.—Brookline is an example of a town that maintains both maps and plans of sufficient detail and excellence for all municipal purposes. The system was laid out about thirty years ago by the late Alexis H. French, M. Am. Soc. C. E., then Town Engineer, and consists of the following: (1) a lithographed topographic map on a scale of 400 ft. to 1 in. with 5-ft. contours, the original of which was platted at 100 ft. to 1 in.; (2) a set of plans on a scale of 50 ft. to 1 in., showing street lines, exact positions of buildings, including porch and bay-window projections, and locations of sanitary sewers and surface water drains; (3) a similar set of plans showing water mains; (4) an equally detailed set of assessors' plans, on a scale of 60 ft. to 1 in., which is brought up to date each year as regards property boundaries and buildings; (5) sewer plats and profiles showing details of construction on 50-ft. to 1 in. horizontal and 5-ft. to 1 in. vertical scales; and (6) a set of street plans showing alignment data. Street lines in Brookline are laid out by tangents and curves marked with heavy cut-granite monuments set flush with the ground at all corners and points of curvature. These monuments serve also as benchmarks. Where buildings and other objects interfere with their true location they are placed on offsets.

This system of maps and plans has proved extremely useful, the 400-ft. map serving as a general map on which to plan, and the 50-ft. maps serving

for layouts. The topographic map, which is of principal interest in this connection, was made by the Town Engineer's staff, working mostly during the winter months when the foliage was least troublesome and the marshes were frozen, this being also the time of year when the services of these men were least required in supervising construction work. The map, at present, is in almost constant use by city officials and real estate developers, and according to Henry A. Varney, M. Am. Soc. C. E., the present Town Engineer, "has proved to be worth many times its original cost." Although the topographic features range from flat marsh lands to steep hills, the 400-ft. scale and 5-ft. contour interval have proved entirely satisfactory. The cost of making the map in pre-war days at 100-ft. scale, was about $2 000 per sq. mile, or more than $3 per acre. Mr. Varney estimates that a survey and map similar in all respects could not be made at present for less than three times this amount. The cost of preparation of the various plans alluded to is not included in this figure.

Richmond, Va.—The City of Richmond furnishes another example of high-grade city mapping accomplished by ground survey methods, but of more recent date than the Brookline example. This survey was completed in 1923 and covers about 31 sq. miles immediately surrounding the corporate limits of the city, at a scale of 200 ft. to 1 in. Contours were sketched with 1-ft., 2-ft., and 4-ft. intervals depending on the steepness of the terrain. It was made by contract and required more than two years for its completion at a cost to the city of $58 000, or nearly $1 900 per sq. mile. The work was done by the most modern methods of ground surveying. The horizontal control consisted of a rigid triangulation net and traverses, the topographic work being done with a plane-table. Altogether, it is a remarkably fine piece of work that has met all the requirements of city planning for which it was intended. Referring to this map, W. F. Woodson, M. Am. Soc. C. E., Engineer-Secretary for the Zoning Commission of Richmond, writes as follows:

"We do not hesitate to use it for the location and paper projection of street, sewer, and drainage systems, and other projects common to municipal engineering; also, the fixing of grade elevations of same and computing of quantities for cost estimating. This is particularly true in the case of requests for appropriation from the City Council. However, under no circumstances would we attempt to use them for the preparation of final construction plans or for computations of pay quantities."

The value of so large a scale as 200 ft. to 1 in. for city planning is clearly shown in this statement. It is to be noted, however, that even this large scale map did not dispense with the necessity of making separate plans on still larger scale for construction purposes. This emphasizes the point previously made that maps and construction plans are not synonymous—neither the one nor the other can be made to serve both purposes.

Aerial Maps of Cities

Compared with the rapid progress that has been made in the use of aerial mapping in other fields, city surveys from the air have been notoriously slow in gaining favor. This is attributable, in large measure, to the inadequacy of some of the earlier efforts and in part to misconceptions of city

officials and engineers of the cost and attainable accuracy. About 1921 several cities in the United States were photographed from the air, but the resulting maps, which were of the mosaic type composed of a patchwork of prints often lacking uniformity of scale and proper ground control, did not prove to be especially serviceable. They were, as a rule, on scales too small to be useful for anything except general reference. These early city maps were largely experimental, the work being done with the undeveloped appliances and methods then available, and frequently with no especial thought of producing maps directly adapted to city planning, or, for that matter, to any municipal purposes. Some of these early maps were made by the U. S. Air Service in the days when the Government favored such endeavor for the sake of keeping its air personnel in training, and when cities were only too glad to obtain so novel a thing as an aerial map on any scale at little or no cost.

About 1923 a ruling of the War Department forbade the Air Service from extending further co-operation of this kind, and since then, municipalities desiring aerial surveys have been compelled to contract with commercial companies for such service. This led to carefully prepared specifications for aerial maps and to earnest consideration of the best types of aerial surveys for city-planning purposes. It was recognized that larger scales were needed and that accuracy was principally dependent on the character of the ground control. The "uncontrolled mosaic" type of map has become practically obsolete and has made place for two other types, namely, the "controlled mosaic" and the "aerial line map".

Controlled Mosaics

Controlled mosaics are continuous photographic reproductions of the ground surface, as nearly as possible free from unsightly matching lines and offsets, such as usually mar the appearance of uncontrolled mosaics. Unlike uncontrolled mosaics they are made by exercising special care to bring all component parts to uniform scale and orienting these parts with reference to accurate ground control. The technique of constructing a high-grade controlled mosaic is by no means simple. It involves a high order of skill that cannot be acquired without considerable training and experience, and this statement applies with equal force to the work of the pilot who handles the airplane, to the photographer's work in the air and in the dark room, and to the civil engineer's work in constructing a scaleable map out of a multitude of photographs all of which are affected more or less by errors in scale, displacements due to perspective, and distortions caused by tilting of the camera at the instant of exposure. Three distinct vocations are thus joined in producing maps of this kind, and it is important that all three be tended by experts if a first-class mosaic is to be produced. Obviously, this is no field for amateurs, nor is it reasonable to expect any one person to become accomplished along all three lines. This fact will account for the scarcity of commercial companies equipped to produce first-class controlled mosaics.

It has become standard practice to take the photographs from high altitudes (8 000 to 15 000 ft. above the ground), in straight parallel lines and with considerable overlap—as much as 60% in the direction of flight and

50% between adjacent flights. After the prints have been made and examined, the scale of each photograph is corrected by reference to ground control, paper control, and by computing scale ratios between successive photographs. With adequate ground control there is no difficulty in producing a controlled mosaic of a city on nearly level ground accurately to scale and free from distortions. Compared with ordinary maps made from ground surveys, the controlled mosaic under such conditions is nearly always superior both as regards accuracy and completeness. Where the terrain is rolling or hilly it is impracticable to avoid slight local displacements due to perspective, and discrepancies in scale due to differences of ground elevation. These troubles are reduced to the minimum by utilizing only the central portions of the exposures, that is, those areas that were most directly under the camera and, therefore, least affected by perspective and by distortion due to tilting of the camera. Nevertheless, not all troubles of this kind can thus be eliminated, which has given rise to the criticism that controlled mosaics are only to be considered as pictures and not as maps. This criticism is entirely unfounded, for controlled mosaics have been produced in which no scaleable errors could be detected (the aerial map of New York, N. Y., for instance), and unjust also because of the tacit inference that ground survey maps are more nearly true representations of the ground surface. The writer's experience with maps of both kinds indicates that ground survey maps rarely possess the accuracy that is claimed for them, and that of the two, the aerial map, on an average, is the more reliable. The reason is that ground survey errors are often cumulative and indeterminate and may affect considerable areas, whereas the errors in controlled mosaics are nearly always purely local and do not affect the accuracy of the map as a whole. This is well illustrated by referring to the leaning of tall buildings in an aerial map. Although the tops of the buildings may be displaced and out of scale, their ground plans as shown in the map are not affected thereby.

The Controlled Mosaic of New York City.—Probably the most conspicuous example of a controlled mosaic of a large city is that of Greater New York made for the Board of Estimate and Apportionment. It covers the five Boroughs, comprising an area of more than 600 sq. miles, and was delivered in two separate forms: A one-piece wall map about 8 ft. square, for general reference purposes, on a scale of 2 000 ft. to 1 in. (1 to 24 000), and a set of 126 sheets, each 14 by 21 in. (35 by 53 cm.) at 600 ft. to 1 in. (1 to 7 200), covering about 400 sq. miles within the official city limits. More than 2 000 exposures, each 7 by 9 in. (18 by 24 cm.), were required for its construction. It was completed in 1924 at a cost of about $35 000 and has been used extensively by the engineers of the Board of Estimate and Apportionment in planning streets in suburban sections, in developing water-front facilities, and in planning the extension of rapid transit lines.

The City of New York did not embark on an experiment when it ordered this map. Previously, its engineers had made use of controlled mosaics of small areas that required intensive study. Among these is a 200-ft. scale mosaic of Coney Island, including the boardwalk, the concessions, and shore

protection; also a mosaic for planning the new Queens Boulevard, and one showing the route of the Belt Railroad.

Perhaps the only criticism that might be made of the New York map is its small scale. No doubt, a 400-ft. scale would have been more advantageous than the 600-ft. scale used. However, the area that had to be mapped was very large and considerations of cost dictated in favor of the smaller scale. Arthur S. Tuttle, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment, advises that the map thus far has served all the purposes for which it was intended.

The Controlled Mosaic of Los Angeles, Calif.—This is a good example of a large scale controlled mosaic of a part of a city. It covers a strip 4½ miles long by 1⅓ miles wide (11 by 3½ km.) in the business section of Los Angeles, and was made in 1922 for the purpose of studying means of relieving traffic congestion. Its scale is 200 ft. to 1 in. The photographs were taken between 11:30 A. M. and 1:30 P. M. of one day, and simultaneously traffic counts and flow studies were made at numerous street corners. The mosaic is in sufficient detail to permit of counting automobiles within any given area, including those within the shadows of high buildings, and to differentiate between stationary and moving vehicles. From the information thus obtained a definite relation was established between the number of vehicles within an area and their rate of movement and number passing a certain street intersection in a given time. The accuracy of this mosaic was such that street widening, elimination of awkward angles and turns, and the possibilities of cutting through new streets could be worked out directly on the mosaic, and the nature of each piece of property affected ascertained from it.

It is significant that this traffic study and aerial survey were initiated and financed by the Automobile Club of Southern California, and not by the City of Los Angeles. It is often better to map a restricted area on a large scale for problems of this kind, rather than to undertake to map an entire city on a scale too small to serve any one purpose well. The work of making the Los Angeles map was contracted for $1 625, which figure did not include the cost of insurance against damage to property assumed by the Automobile Club. The amount, $1 625, is not representative of the cost of surveys of this kind and the writer has learned that the parties who made the aerial map lost considerable money on their contract.

Aerial Line Maps

For most city planning purposes, and especially for zoning, line maps are preferable to controlled mosaics. They are prepared by transferring to tracing linen by hand, with ink, the more important features portrayed in the photographs, the final product being in all respects similar to maps prepared by the standard methods of surveying and drafting, but usually more faithful and complete in the matter of detail. Contour lines may be added by taking into the field a line map mounted on a plane-table, or better, by using the aerial photographs themselves as plane-table sheets and platting the elevations and contours directly on them and then transferring the contours together with other features to tracing linen. The latter method is

especially recommended in suburban districts where few buildings obstruct the surveyor's view and where stadia measurements may be readily taken. With the photographic detail before him the surveyor orients his plane-table conveniently, finds his work of sketching contours greatly expedited, and the accuracy generally improved.

Besides affording a clearer base map for planning and zoning purposes, the line map excels the controlled mosaic in other respects. It can usually be made more accurately because all the distortions and displacements alluded to as occurring normally in controlled mosaics, especially in hilly country, are eliminated in the geometrical processes through which a line map must pass in its evolution from the original aerial photographs into its final form. One process that has found wide application for this purpose and has proved simple in technique and reliable by being self-checking, is that known as the method of radial control. It consists, in essence, of a graphical triangulation worked out from the photographs themselves, using the central points in the latter and lines connecting these centers as a base.* Space does not permit of describing the radial control method more than to state that it effectually determines with accuracy the location of important points at any desired intervals throughout the map. Connection is thus established with control points on the ground so as to maintain true scale, and also the intervening detail may be referenced to the points located by radial control. If these are spaced from 1 in. to 2 in. apart, as is usually the case, the map features are tied in a manner that precludes errors from becoming cumulative. The accuracy with which the radial control points are located corresponds to the width of a fine pencil line or as close as a draftsman can work.

The earlier attempts at producing line maps included, among other things, sketching details in ink on the photographs and bleaching out the superfluous detail with chemicals. This method has been discarded because it failed to correct displacements and distortions and produced maps containing the same local errors that are found in mosaics. For the same reason it is not advisable to prepare a line map by tracing from a controlled mosaic. The work of tracing should always be done directly from the original or enlarged photographs, making adjustments and corrections as needed.

Accompanied by the original photographs from which it is made, a modern aerial line map furnishes a scope of information that cannot be equalled or approximated by any other known form of survey or map. The line map itself is a most convenient base for a variety of purposes, on which to enter statistics, zoning symbols, and shading, and to project new thoroughfares and other improvements. The aerial photographs are a permanent source of information for details covering a wide range of interests. They may be used for studying vegetation, such as shade trees for parkways; the location and extent of different kinds of surfacing used in street paving; the nature of buildings; conditions of lots and back yards; exact extent of improvements made in new sub-divisions; all that pertains to traffic conditions, both foot and vehicular; parking spaces in streets, yards, and vacant lots; railroad terminal facilities; and harbor and water-front problems.

* For a description of the method of radial control, see *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 792-799.

In order to derive the maximum value from aerial photographs for such studies they should be viewed with the aid of a specially constructed stereoscope. It is necessary to view two different photographs at one time, and portions of these photographs must cover the same ground area. The area under observation appears in relief, and when the stereoscope is properly adjusted this relief is seen with astonishing clarity. Hills, mountains, woodland, houses, and even such minor features as fences and single trees, project upward as if endowed with a third dimension. Valleys, ravines, and even the smaller gulches, appear as if carved into the earth's surface. When viewing parts of a city this third-dimension effect is of the greatest assistance in judging the relative heights of buildings, the general drainage and topography, and in differentiating between trees and shrubbery. Church steeples, factory chimneys, and other tall slender objects come into prominence that otherwise would escape notice. Actual differences of elevation cannot be determined through mere stereoscopic vision, but a good conception of relative differences of elevation may be gained.

In many respects, aerial photographs viewed in this manner, in connection with a line map, are vastly more useful than a complete controlled mosaic. The stereoscope reveals topography to the layman's eye in a manner that contours or hachures cannot accomplish. The combination of line map and photographic prints is, therefore, especially suited for use before commissions composed of non-technical men. The use of the stereoscope will frequently obviate the necessity of contouring a map.

The Line Map of London, Ont., Canada.—This very excellent map was produced by co-operation between the City of London and the Geodetic Survey of Canada. It consists of 57 lithographic sheets each containing an area measuring 3 000 ft. north and south by 2 000 ft. east and west, on paper cut 29 by 40 in. The scale of publication is 1 to 1 000, or 83⅓ ft. to 1 in. Contours are drawn at 2½-ft. intervals. A remarkable scope of information bearing on municipal matters is embodied in this map, including, in addition to the usual features, such as street, sidewalk, and building outlines, the position of all bench-marks, trees, catch-basins, manholes, valves, hydrants, electric light, telegraph, and telephone poles, and even fire-alarm and mail boxes. Conventional signs are used to denote these various smaller features, supplemented by a legend. Four colors are used, namely, black for the main map features; burnt sienna for contours; green for trees; and blue for watercourses.

The map was made primarily by ground-survey methods and is of interest because aerial photographs were used for checking up the outlines of buildings, curbs, and other important features. The photographs were taken from an altitude of about 5 000 ft. and required enlargement of four to five diameters in order to make direct comparison with the map possible. The photographic projection was rectified to fit the concrete sidewalks surrounding each block, as nearly as possible. An accurate check on the position of the detail in the interior of each block was thus obtained.

No cost figures are available for publication. As the work was done partly by the city and partly by the Geodetic Survey, which included during the

summer months a number of college men who had to be familiarized with this new form of surveying, it is natural that the cost should be high and afford no criterion by which to judge the cost of similar surveys made by a completely trained organization.

The scale of the London map seems unnecessarily large. It is believed that a scale one-half as large, or about 200 ft. to 1 in., would have shown all that the existing map shows with equal clearness and without crowding. This would have made an appreciable difference in cost, not the least part of which would be the reduction of the number of sheets to one-fourth. It would have been more useful for city planning purposes by giving a more comprehensive view of the city. At the present scale, the precise geodetic control data do not show for what they are worth, but are an asset to the city in all future surveys, especially those in which a highly precise base is of fundamental importance.

Control and Accuracy

These two subjects, being in large measure interdependent, are best considered under one head. Other things being equal, the accuracy of an aerial survey depends on two factors, namely, the character of the ground control and the permissible cost. Given sufficient latitude in these two respects, any reasonable limit of error is practicable of attainment. The question as to what degree of accuracy to provide in a given case is not to be gauged solely, however, by these two considerations. The following practical aspects, which are often overlooked, have a fundamental bearing on the matter. There is a limit to the accuracy with which map features can be delineated. The controlled mosaic is superior in this respect to the line map. In the latter the ink lines have an appreciable width, and if drawn out of position by only their width, which would not be considered a great defect in some drafting-rooms, the error thus introduced is quite considerable. At a scale of 400 ft. to 1 in., for example, an ink line on tracing linen may easily have a width of 4 to 5 ft., and in the course of preparing the map, expansion or shrinkage of the linen may displace such lines by an equal amount. To attempt to measure distances on such a map nearer than 5 or even 10 ft. is merely deluding oneself. In a distance of a city block, say, 500 ft., an error of 10 ft. in scaling would represent 1 part in 50. On the same map, a longer distance, say, 5 000 ft., may be scaled with no greater absolute error, and this would represent an accuracy of 1 part in 500. This same map may be based on control consisting of a triangulation having errors not exceeding 1 part in 100 000, supplemented by control traverses having errors not exceeding 1 part in 20 000. From these considerations it may readily be seen that a highly precise and, therefore, costly control is not warranted where merely a map for general purposes, such as city planning, is required, because to the map user who is concerned only with the detailed features and not with the triangulation or traverse points, which are rarely shown on any finished map, this great precision would be completely lost. Only where a city needs a cadastral survey in order to fix the exact positions of high-priced property boundaries is a geodetically precise control worth its cost.

The best form of control for use in constructing aerial maps, whether of the mosaic or line-map types, is a network of closed traverse lines established with transit and steel tape. The distances between traverse lines may be varied to suit considerations of cost and accuracy. On fairly level ground, traverses at ½-mile intervals, and even 1-mile intervals, will suffice, the graphical control derived from the aerial photographs being amply accurate to reference objects into correct position in the intervening spaces. For rolling terrain, the traverses should be at shorter intervals, and for very rough topography every street may require a traverse. It is not necessary to establish this control before taking the aerial photographs. Often, it is better to make the exposures first and then to select from the photographs objects such as edge of curb, street-car rail intersections, hydrants, etc., that permit of ready identification both in the photographs and on the ground and that will serve as control points where most needed in the construction of the map. Because of the overhang of roofs and photographic displacements of tall buildings, traverse lines should not be made to follow building lines. Edges of curbs and street-car rails are the most convenient lines to follow. In any event, traverse points should be referenced to a system of co-ordinates before platting. This serves to eliminate errors in drafting, and makes the traverse data of value for use in other city surveys.

With reference to accuracy in city planning maps, so eminent an authority as the late Nelson P. Lewis, M. Am. Soc. C. E., made the following statement in a paper on aerial surveys:*

"Great accuracy is unnecessary during the earlier stages of city planning. It might be well also to emphasize the fact that in planning new territory it is a mistake to make the first plans in great detail. The essential thing is the general character of the plan which should be worked out in a logical manner, and if that be done with the existing topographical conditions as a basis, the plan cannot well be spoiled in the subsequent elaboration of detail."

Photographic Enlargements

Photographic enlargements are of material assistance in making detailed studies of given areas, and are best made directly from the original negatives. For some purposes a part of a completed map may be enlarged satisfactorily by rephotographing on an 8 by 10-in. or 14 by 17-in. plate, the negative being placed in the enlarging camera. It is not advisable in such case to enlarge more than two diameters, as greater ratios cause the definition to become impaired. Working from the original negatives greater ratios are feasible and enlargement to as much as eight diameters has been used. However, the fuzziness of the resulting prints detracts from their appearance and does not promote accuracy or ease in scaling dimensions. The only advantage gained is in producing maps large enough to permit entering the names of lot owners, dimensions, and other data. A safe rule is not to enlarge more than three diameters if a good clear map is the objective.

Considerable cost may be saved by making the exposures at small scales, such as 800 or 1 000 ft. to 1 in., and constructing the map by enlargement

* "A New Aid in City Planning," *American City Magazine*, March, 1922.

at 400 ft. to 1 in. As the number of photographs required to map a given area varies inversely as the square of the scale, proper enlargement will often be the logical solution for keeping the cost within budget limits.

Reproduction

This is an important subject, because the city planner or zoner usually requires many copies for his work. Blue-printing makes reproduction a simple matter for line maps on tracing linen. Mosaics, on the other hand, require reproduction by photographic process, and this usually costs so much as to place restrictions on the number of copies that can be used. Where large numbers of copies are desired, the photo-gelatine process should be resorted to. This requires the making of a special negative, but brings the cost per copy down to a nominal price.

Ordinarily, mosaics are reproduced by copying sections on the largest commercially made negatives. These measure 20 by 24 in., and it is possible to make blue-prints from these negatives. Such photographic blue-prints often lack definition, however, and are not to be recommended except for preliminary studies or sketching. Copying a mosaic on small negatives and making enlarged prints from the latter is being done successfully and saves cost. It is to be hoped that as time goes on, and aerial mapping processes are further developed, better and cheaper processes of reproduction will be evolved.

Cost of Aerial Maps

Many city planners and engineers have been disappointed at the high cost of aerial surveys, because most people are inclined to under-estimate the difficulties involved in aerial map construction. As has been stated, special appliances and highly trained personnel are required in this work, and the taking of photographs is limited to perfectly clear days. A suitable day occurs on an average of once in seven for the greater part of the United States.

The principal factors affecting cost are: (*a*) scale; (*b*) size of area; (*c*) shape of area; (*d*) distance to nearest landing field; (*e*) degree of accuracy required; and (*f*) amount of enlargement, if any. Contract prices for aerial maps are as a rule lump sum figures in which all the factors surrounding a given case have been taken into account. Prices per square mile or per acre are rarely quoted. Ordinarily, ground-surveying operations to establish control and for locating contours are executed by the city engineer's staff. If the staff can handle such work to advantage, cost may be saved over having these phases of the work done by contract. It will be seen from the foregoing considerations that it is not practicable to state definite costs of aerial maps. Roughly speaking, first-class controlled mosaics of cities cannot be made for less than $100 per sq. mile. The various factors mentioned may affect the cost several hundred per cent. In some cities the difficulties of obtaining clear pictures on account of smoke or dust are considerable, and this will materially affect the cost.

In spite of their high cost, aerial surveys are appreciably cheaper than ground surveys, assuming the same accuracy and completeness of detail in

either case. They possess the inestimable advantages of saving a great deal of time over the older standard methods of surveying and drafting, and furnishing a completeness of detail and information that is not obtainable by any other known processes. An aerial map of a city, once properly made, is easily kept up to date by re-photographing from time to time such areas as are rapidly developing, and a permanent record is thus obtained that has a distinct value of its own for statistical and legal purposes.

Oblique Aerial Photographs

Taken in conjunction with mosaics or line maps, oblique photographs (taken at an angle with the vertical) are of decided value in city planning work. They are usually taken from airplanes flying at comparatively low altitudes and show a surprising amount of detail relating to character of buildings, such as number of stories, whether residence, store, office, garage, or factory, that cannot readily be gleaned from vertical aerial photographs, such as are used in the construction of maps. Oblique photographs are, of course, not scaleable, but have the advantage of covering large areas and being inexpensive. For use before commissions and public hearings they are of great practical value. In connection with the planning of parks and public playgrounds, or wherever landscape architecture is required, they are more useful than maps.

DISCUSSION

ARTHUR S. TUTTLE,* M. AM. SOC. C. E. (by letter).—This paper appears to cover the subject so thoroughly as to leave little opportunity for further amplification or discussion. It may be advisable, however, to emphasize a number of points to which the author refers and to explain a little more fully the reasons that determined the selection of the 600-ft. scale for the New York City air map.

While the question of cost was, of course, an important consideration, the prime factors in the determination of the scale to be adopted were the utility and convenience of the completed map. The only official map of the entire city, completed in 1915 and revised systematically to date to show all adopted map changes, is on a scale of 600 ft. to 1 in. and comprises 35 sheets or sections, each measuring 29 by 42½ in., including a 1-in. lap. This scale has been demonstrated by long experience to meet all the requirements of a general map and to serve as an index of the large scale plans on which the details of each individual map change or project are shown. It seemed obvious that the usefulness of an air map in the visualization of city problems would be greatly enhanced if it could be studied in connection with a line map of corresponding area, and that for this purpose there would be a distinct advantage in having the scales of the two maps identical. In the writer's opinion, this contention, which largely determined the adoption of the 600-ft. scale, has been conclusively demonstrated to have been sound, and in view of the size of the area covered it does not appear that any advantage would have resulted from using a larger scale.

The impracticability of reproducing or handling photographic copies of the air map, measuring 29 by 42½ in. in size, led to the plan of subdividing the area covered by each section of the city map into four sub-sections, each bearing the same serial number as the corresponding map section, to which was added a distinguishing letter to complete the identification of the individual photographic map sections. It may be of interest to state that the mounted sectional photographic map sheets make an atlas measuring 2½ in. in thickness, without covers.

With regard to accuracy, the contract under which the New York City air map was furnished provided that measurements made between any two points on the map not less than 9 in. (5 400 ft.) apart should not vary more than 3% from the corresponding distance on the control system. In this case the control was the city map previously referred to, the groundwork of which is based on a precise triangulation survey executed between 1903 and 1908 jointly by the U. S. Coast and Geodetic Survey and the City of New York. Before the air map was accepted it was seen that this provision of the contract had been rigidly complied with and a more recent test involving the comparison of sixteen scaled distances, ranging from 5 500 ft. to about 6 400 ft. in length, has indicated a maximum discrepancy of only 1.6% and an average of 0.6 per cent.

* Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

In discussing the question of cost the author notes the requirements relating to special appliances and highly trained personnel and calls attention to the fact that the taking of photographs is limited to perfectly clear days which, he states, occur on an average of 1 day in 7, or about 15% of the time, for the greater part of the United States. In partial confirmation of this condition it may be stated that between August 9 and October 17, 1923, when the New York City area was photographed, there were only 11 days when satisfactory atmospheric conditions prevailed and that it was subsequently found necessary to re-photograph a part of the Manhattan area to secure clearer definition. It should also be remembered that photographs must be secured toward the middle of the day in order to minimize the effect of long shadows, and that for heavily wooded areas the topographical features can only be secured at a time of the year when the foliage is not dense. The last consideration will not generally apply in the case of city work.

A carefully made air map is of inestimable value as an adjunct to the visualization and solution of city planning problems. In some instances where other maps or data are lacking and the time element is the all-important consideration, photography supplies a means which has hitherto been lacking for securing fairly accurate and detailed information with the greatest possible dispatch and economy. In general, however, such maps made entirely by photographic processes will not supplant the draftsman's portrayal of the situation, based on accurate and detailed ground surveys, but will be used rather to supplement and clarify the picture.

F. L. Olmsted,* Esq. (by letter).—The writer can only confirm in general all that Mr. Matthes has said regarding aerial surveys for city planning, especially his statement of the advantages of using the original aerial photographs in connection with a line map.

On one occasion the writer had made arrangements for the preparation of a controlled mosaic map covering about 250 sq. miles, as a basis for a preliminary general city plan, but owing to bad weather and other adverse conditions only the original unadjusted aerial photographs were ready at the time it became necessary to push the field studies for the plan. The mosaic was to have been enlarged about two and one-half diameters from the scale of the original photographs and to be on the scale of 400 ft. to 1 in. The photographs, varying considerably in individual scale but averaging about 1 000 ft. to 1 in., were matched together in long strips of overlapping prints fastened by adhesive paper "hinges". Many section corners and section lines were recognizable by the streets, roads, and fence lines. Large sheets of tracing cloth were ruled off into mile squares at the scale of 1 000 ft. to 1 in., to represent the approximate locations of section lines, although these were known, of course, not to be exactly mile squares. With these as a rough control a hasty tracing was made from the photographic strips showing the main features—roads, ponds, streams, etc.—as a general rough guide map.

Field studies were then made using the strips of original photographs, folded accordion fashion and carried on a strip of "compo-board" held by

* Olmsted Brothers, Landscape Archts., Brookline, Mass.

rubber bands, prints of the general guide-map tracing being carried along for reference. Field notes were made in ink and in pencil directly on the photographic prints, the wealth of detail—trees, fences, buildings, roads, trails, ponds, and streams—sufficing to locate on the prints instantly and with precision (in relation to surrounding objects) any point observed on the ground. Elevations were spotted on the photographs at the principal points along lines where vertical control had been run by field parties. The variations in scale between adjacent photographs of course made it impossible to record without danger of considerable distortion alignments extending across more than one photograph and not represented on the ground by visible objects, such as roads or fences, and the map was wholly without contours.

As nearly all the topography was very gentle and the lines of natural drainage were well indicated by brooks and ditches, it was possible, subject to later adjustment of alignments of proposed streets extending across the matching lines of the photographs, to complete nearly all the field work for the general plan before making either a controlled mosaic map or a properly controlled line map. The problem of subsequently transferring and adjusting the details of the general plan to the completed controlled map at uniform scale presented no serious difficulty, as the plan was practically complete in detail and closely tied at all essential points to recognizable objects, with a local error kept within a limit of less than 20 ft. wherever that degree of precision was important.

This "cart-before-the-horse" method should be adopted only where considerations of speed absolutely necessitate it; but the writer's experience with it strongly emphasizes the advantages of working directly on the original aerial photographs with their complete detail, in conjunction with a line map based on those photographs and showing only a small fraction of their detail but correct in scale. The occasional use of a magnifying glass, and, for certain purposes, of a stereoscope, with these small-scale photographs and the taking of some pains in drafting and note-making, permit the accomplishment of better results in less time than is possible by ordinary methods with maps on a much larger scale. If time and funds permit, it is easier to work out such plans on maps at a scale of 400 ft. to 1 in.; but by taking the necessary pains in the planning work equally good results can be produced at a greatly reduced total cost for mapping and planning if done at a smaller scale. However, 1 000 ft. to 1 in. appears to be near the limit of workability.

Issue can be taken with Mr. Matthes on only one point. He is quite right in stating that horizontal control of a geodetic precision is wholly unnecessary for maps intended as a basis for general city planning; but it is equally true, as he also indicates, that such general plans must be followed in course of time with large-scale precise fragmentary plans for construction, and for cadastral purposes. In the long run, also, it saves a great deal of trouble for the city's engineering department and for landowners if the principal monuments, to which both fragmentary cadastral surveys and the city's fragmentary detailed construction surveys must be tied, are located once for all in relation to a single uniform system of co-ordinates by means of a control survey of a high

degree of precision. The making of a general map of the whole territory, even if its scale is to be so small that a non-cumulative error of 4 ft. is wholly negligible, necessitates some general system of horizontal and vertical control for the whole territory and makes an occasion for getting a main skeleton of primary control sufficiently precise to be used for all subsequent detailed surveys on any scale, so that as they are gradually extended they will fit without "fudging" where they meet and overlap.

Charles N. Green,* M. Am. Soc. C. E.—In a study of Northern New Jersey with the idea of betterment of transit facilities, air pictures were made of an area of about 150 sq. miles. From east to west the area extended approximately from the Hudson River to the Orange Mountains, and from north to south from Paterson to and including Newark. All the photographs were made oblique, and were taken at an elevation of about 10 000 ft. The photographs, of course, were not in detail. If a particular area were required in detail, as, for example, that point where the Erie and Lackawanna Railroads cross before entering the tunnels in Jersey City, and also some entire towns, the prints were enlarged.

That particular point—the Erie-Lackawanna crossing—is a most difficult spot to photograph. Apparently, it can only be done on Sunday morning with a northwest wind. This crossing, not at grade, with more than fifty trains passing over the two roads on a week-day morning, together with moisture in the air, bring about an atmospheric condition so dense that it cannot be penetrated even with a screen. Several attempts were made and none of them was satisfactory.

About 40% of each photograph is all that can be used; the remainder has to be covered by the overlap on the next one. The photographs were taken in long sweeps across the area, probably one every mile or two so that the area was fairly well covered. This method, however, requires a location map to show approximately the area covered by each photograph. With this ready means of reference, any area may be picked out and studied in detail, with the further aid of an enlargement if desired. For the purpose for which these photographs were made, they have been very satisfactory and have saved an incalculable amount of time.

Theron M. Ripley,† M. Am. Soc. C. E. (by letter).—The aerial survey and map which is being made of Erie County, New York, is primarily for the purpose of the general study of the Greater Motorway System and preliminary ground location of the various roads comprising that System. The survey was authorized by the Board of Supervisors and ordered by the County Engineer, George C. Diehl, M. Am. Soc. C. E.

As the first order for this work covers the City of Lackawanna, extending a short distance into the City of Buffalo, and, later, is to cover the entire city, it may be apropos to discuss Mr. Matthes' paper, at least in part, from experience gained in this work.

* New York, N. Y.

† Chf. Engr., Greater Motorway System of Erie County, Buffalo, N. Y.

To one desiring to secure an aerial survey of his city or county the writer would say "do not be discouraged if your officials do not immediately accept your recommendations and make the appropriation to carry them out". It will be necessary to "sell" them the proposition, unless by rare good fortune they are already "sold", as the survey will seem (to them) of far less value than it is; moreover, the amount of money requested will look much larger in a lump sum, than if the same or a larger amount was paid out in monthly pay-rolls. The "selling" campaign should be kept up, however, because in no other way can the same amount of detailed information be acquired in so accurate a manner.

When one mentions a cost of $100 per sq. mile it does not sound very large, but as soon as it is multiplied by 200 or 1 000, the impression is entirely different. The latter is exactly what engineers must do if they enter on a large city or county proposition and desire to have the plans as their exclusive property.

Aerial surveys have been made of large areas and the ownership of the negatives left in the possession of the company that did the work, the client thereby receiving his maps at a greatly reduced price. In Erie County this is not believed to be good practice. Therefore, the work is the exclusive property of the County. Any engineer or board working on a general planning scheme will easily realize how his or its work can be hampered by conflicting opinions arising from a general distribution of the survey maps before plans are perfected and, therefore, the great advantage of curtailing such distribution. Such conflict could easily cost the city or county many times the difference in cost between private and general distribution of maps.

By clientele ownership, as outlined, the writer does not mean that any one but the board of officials directly responsible can have access to the maps; in fact, the officials of the Greater Motorway System are proud of their aerial survey and map and are only too glad to show it to any one and furnish any information relative thereto, but they must come to the office for such examination.

The aerial work of this survey was done by the local firm of Ronne and Washburn and all the indoor work, from developing the negatives to the finished map, by the Fairchild Aerial Surveys, Inc., of New York City, while the ground control surveys were made by the Engineering Staff of Erie County. The flying was done at an altitude of 9 600 ft. and the finished map is 600 ft. to 1 in.

The work is a controlled mosaic and, with few exceptions, control points are 2 miles or less distant from each other. The traverses of existing roads were used for the control. Most of these roads have been surveyed for existing highway improvements and the notes of these surveys were used in plotting the control; nevertheless, it was necessary to run 40 miles of new traverses. This field work and the plotting of the total control of 121 miles cost $1 100, which includes only the cost of paper, tracing cloth, and the salaries of the men actually employed on the field and office work; in other words, no overhead of any kind.

Had it been necessary to traverse the entire 121 miles it is estimated this cost would have been at least $2 100. If this work had been done by inexperienced men, the cost could easily have been 50% greater. As stated by Mr. Matthes, too much time and money should not be spent on undue refinement for accuracy of control. This accuracy should depend on that desired in the completed map. The platting of a control map in the ordinary drafting room, when such map is to cover 170 sq. miles of territory on a scale of 8.8 in. to the mile, is quite a task. The sheets are large and difficult to handle and to retrace the work for the purpose of closing an error in traverse may be time and labor wasted.

In attempting to close traverses several miles in length it is readily seen that a failure to close when the error is less than 1% is, for all practical purposes, accuracy. One of the traverses on the work described, 13 miles in length, showed just such a difference with the result that a closure was effected by "fudging", namely, swinging the tracing about a selected point of the original platting and thereby saving several hours of work.

The map was made on the basis of a variation of not to exceed 1% between the control map and the aerial map, with points on the former not more than 2 miles apart. In other words, the aerial map had to scale the same as the control map up to 8.8 in., with a variation not to exceed 0.088 in., that is, scaling 1 mile each way from control points 2 miles apart.

Seven courses taken at random from the first map received and varying in length from 4 960 to 11 640 ft. on the control map, showed a maximum variation of 1.55% on a course 5 780 ft. long and a minimum variation of zero on a course 4 960 ft. long. The sum of the several courses as scaled on the control map equalled 52 410 ft. and on the aerial map, 52 490 ft., that is, the errors were both plus and minus.

For the purpose of general study and planning it is at once apparent how trivial 1% or even 2% in variation of map distance becomes when the error is not cumulative and when such error does not exceed 0.1 in., or even 0.2 in., in open country. Where the control points can be closer together, as in street areas, this error can be reduced to 0.01 in. on a 600-ft. block, closer than the draftsman can plot it.

The map of Erie County is being furnished in atlas sheets covering an area of 2½ miles east and west and 1½ miles north and south between border lines. This produces sheets 17 by 27½ in., including the binder edge. The map is also being delivered in sheets covering about 60 sq. miles each. The former is for use in the field and office and the latter for office use only.

The ability to lay out preliminary lines is only one function of an aerial map; in fact, in country similar to that around Buffalo the ability to show a property owner where the line is located, and why it is so located, is of more value than the mere location of the line. For these reasons the securing of rights of way becomes a prominent function of these maps.

During the past summer (1926) a serious accident happened near one of the town parks, and a suit for damages was brought against the Town Board. Testimony was given by both sides that the accident happened at

or adjacent to a certain depression. The County Engineer had furnished the town officials with an enlarged aerial map of the locality in question. This map was entered as evidence, the depression was located thereon, and shown to be off the property owned by the town. The plaintiff lost.

Several statements or suggestions should be emphasized to those about to secure aerial surveys. As Mr. Matthes states, an aerial survey, or general plans therefrom, cannot take the place of ground surveys and detailed plans for construction and pay-estimate purposes. Further, "controls" need be no more accurate than the purpose for which the map is to be used and money should not be spent to secure a greater accuracy.

An aerial map is a picture to scale of all the camera sees; and the limit of its usefulness is only determined by the activities of the locality mapped and the people using it.

Obliques.—For purposes of general discussion before boards and commissions oblique aerial photographs are frequently of more value than an aerial map. This applies with equal force when showing the general layout of a property, such as a subdivision, with any included structures as buildings, bridges, etc.

The oblique is a photograph only and must not be confused with the map; because it is a photograph and shows the terrain in perspective, as is the case in all pictures, the general observer is able to get a clearer idea of the relative locations of points than when they are shown to him only on a map.

Moreover, the low altitude at which obliques are usually taken permits of showing in greater detail matters which are obscured in the higher flights. Also, the effect of presenting a beautiful photograph for the consideration of non-technical men who are passing on a proposition should not be overlooked, and an oblique is just such a picture.

Gerard H. Matthes,* M. Am. Soc. C. E. (by letter).—Among the many valuable points brought out in the discussion of this paper a number merit particular attention.

Mr. Ripley points out the desirability of a county retaining possession of the negatives of an aerial survey made for it by contract. The disposal of negatives has been a much mooted question ever since aerial surveying expanded into an industry. Retention of aerial negatives by the contractor is predicated on the well-established custom of a photographer retaining ownership of the negatives of photographs which he makes for his client. Some aerial mapping companies regard the negatives as the property of the client but to be retained by them for subsequent use subject to the orders and wishes of the client.

Aerial mapping contracts sometimes specify reimbursement to the client of part of the profits derived by the contractor in selling copies of maps or pictures from the client's negatives. In the case of municipalities this practice is not to be recommended Delivery of the negatives would seem an essential to a complete discharge of the terms of any public contract. A precedent

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for this has been established by the contract for the aerial map of New York City let by the Board of Estimate and Apportionment, which called for the delivery to the city of all negatives, including the original negatives as well as the large copy negatives from which the individual map sections were made. The wisdom of this procedure appears to be confirmed by the action taken by Erie County.

On the subject of ground control for aerial surveys for city planning purposes, the point raised by Mr. Olmsted is well taken, namely, that it will often be advantageous for a city to establish a geodetically precise horizontal control once and for all, to serve as a base for future surveys, including those for cadastral and construction purposes. However, as few city engineers are in position to undertake such geodetic surveys for lack of specially trained personnel and special instrumental equipment, such work is usually best done by contracting with properly equipped and qualified firms engaged in such business, or else, by enlisting the co-operation of the U. S. Coast and Geodetic Survey. This Bureau, in the past, has performed valuable service in extending geodetic surveys over certain city areas. Unfortunately, the limited personnel available for this work makes it impracticable for the Coast and Geodetic Survey to undertake co-operative surveys of this kind to any extent.

The discussions have emphasized the practical value of oblique aerial photographs and controlled mosaics, two phases of aerial surveying that are not receiving the recognition that is due them on the part of many engineers, who are prone to class them as mere pictures and picture maps. The fact is that already these two forms of aerial photography have been used effectively in a surprisingly large number of instances where formerly line maps would have been demanded. For certain purposes they have proven so satisfactory (and better than line maps) that the demand for them on the part of engineers and others is constantly increasing. This is especially true in the case of surveys for city planning.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1631

PRODUCING CONCRETE OF UNIFORM QUALITY*

By Roderick B. Young,† Esq.

With Discussion by Messrs. T. P. Watson, Harry C. Boyden, William H. Adams, Thomas K. A. Hendrick, John G. Ahlers, William M. Hall, and Roderick B. Young.

Synopsis

Theoretically, the quality of a concrete can be controlled by its water content but practically this is not sufficient, for concrete is the product of a manufacturing process consisting of several operations, and the control, to be successful, must take into account each of these and provide the proper conditions for their performance. It is the object of this paper to point out what these conditions are, the methods by which one organization has attempted to provide them, and what led it to adopt these methods.

The operations involved in making concrete are (1) selection of the raw materials; (2) design of the concrete mixtures; (3) handling and measurement of the constituent materials; and (4) manufacturing the concrete. Each operation will be considered in turn.

Selection of Materials

The selection of materials is largely an economic problem, for concrete of any desired quality can be made from many different cements and aggregates, provided the proper combinations of these are chosen. Uniformity of materials, however, is of considerable help in controlling the quality of con-

* Presented at the meeting of the Structural Division, Montreal, Que., Canada, October 15, 1925.

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crete and, although control can be obtained when the materials vary, it is obtained more easily and cheaply when they are uniform. For this reason it is desirable to limit the supply of each to one source and to choose those known to vary least, even when this involves some additional expense.

Design of Mixtures

Properties Required.—Having selected the materials, the next step is, naturally, to determine the proportions in which these materials must be combined to give a concrete of the desired properties; but before this can be done, the desired properties must be known and must be measurable. Different conditions of service require concrete possessing different properties to different degrees; in some cases compressive strength is the prime consideration; in others, water-tightness; and, in others, resistance to wear or weathering. A concrete may possess all these to a high degree, but again one or more may be almost entirely absent. The presence of one is no guaranty of the presence of any other.

Of these properties, compressive strength is measured most easily and cheaply and therefore the practice has arisen of using it to rate the quality of a concrete; consequently all engineers design for compressive strength and this in turn has caused compressive strength—particularly that at 28 days—to be considered a measure of all that is desirable in a concrete. This situation should always be borne in mind in designing a concrete mixture, for while it is necessary to rate a concrete by its strength, the design of the mixture should not be based on strength alone unless strength is the only consideration. If other qualities are important, the engineer should make sure that the proportions set will give the qualities desired.

In designing concrete mixtures, there are two fundamental conditions that must be met. The first is imposed by the requirements of the structure for which the concrete is to be used, and is a property of the finished concrete, as, for example, strength, impermeability, etc.; the second is imposed by the methods of manufacture and is a property of the concrete mixture, namely, its plasticity or workability—a transient quality which it needs to possess only during the period of deposition. The two properties are not directly related, workability being no indication of strength or strength of workability. Hence the problem is to design a concrete mixture that will have the properties required by the designer of the structure and at the same time be workable until it is in its final resting place.

Control by Water-Cement Ratio.—Several methods based on different theories have been advanced for the proportioning of concrete mixtures, but as yet the only practicable basis for design is the water-cement-ratio-strength relationship developed by Duff A. Abrams, M. Am. Soc. C. E. Professor Abrams has shown that for concretes required to be workable when moulded, the compressive strength is related to the ratio of water to cement in the freshly placed mixtures, that is, to their water-cement ratio (the Hydro-Electric Power Commission of Ontario expresses this in terms of weights), each such ratio having a corresponding compressive strength. This relation-

ship holding, it follows that if it is desired to obtain a concrete of a certain compressive strength, it is only necessary to know the proper water-cement ratio for that strength and to proportion the mixture of cement, aggregate, and water to give a workable combination having that ratio. Likewise, in attempting to control concrete mixtures, the basic problem is to obtain the proper water-cement ratio and workability simultaneously and continuously.

Handling and Measurement of Materials

Uniformity.—The method of handling the materials prior to measurement is not of great effect on the quality of the concrete except that any method which allows segregation of the aggregate makes it more difficult to obtain uniform concrete. Theoretically, it is possible to compensate for any variations in the aggregates, and thus maintain uniformity in the concrete mixtures, but practically this is not always possible, especially when the materials vary rapidly. It is better to prevent these variations by careful handling than to depend on any corrective measures applied later.

A uniform mix is the basis of a uniform concrete and is obtained only when the same absolute volume of cement, aggregate, and water is in each and every batch. With water the absolute volume is, of course, the same as the measured volume but with the cement and aggregate it is not the same since both as measured contain air spaces. A little consideration will show that the finished concrete, to be uniform, must contain the same quantity of solids per unit volume, while the wet mixture from which the finished concrete is made must contain not only the same quantity of solids per unit volume, but also the same quantity of water. Such a concrete, if not otherwise abused, would be uniform both as to strength and other properties, and, in addition, be easier to control than one with varying quantities of the constituent materials.

Controlling the Quantities.—Measurement of the cement is not usually a problem of control. The packages are of uniform weight and form a convenient and satisfactory unit for measuring the absolute volume of cement. When shipped in bulk, special provision for measurement has, of course, to be made and weighing seems to be the most satisfactory.

The measurement of the water is so intimately related to the measurement of the aggregate that both must be considered together. The goal sought is to obtain the same total volume of water in each and every batch, which total volume is made up of water added plus the water contained in the aggregate. Thus, whereas the water to be added, if known, can be measured with precision, the proper quantity to be used can only be determined after the moisture in the aggregate is ascertained.

The aggregate is the most difficult of the constituents of concrete to measure. The volume of solids in any unit depends on the method of filling the container, the grading of the aggregate, and, in the case of fine aggregates, on the moisture content as well, and these factors act in such a way that their effect can be predicted only approximately.

Measurements by Volume and by Weight.—In the usual method of measuring by volume, there are variations due to all these factors and an added variation due to errors by the workmen in determining when the measure is filled to the proper level. Seldom is any pretense made of leveling a measuring hopper to a mark, and struck measurements are even more rare. In the writer's experience, these several causes combined can produce variations in the volume per batch of wet concrete of from 10 to 20 per cent.

The only uniform features of concrete made in this way are the quantity of cement per batch and the water added. The total water per batch and hence the water-cement ratio and compressive strength will vary. What is equally important, the consistency also will vary; for if the small batch is of the proper consistency, the large batch will be too dry; or, as is more likely, if the mixture is adjusted so that the latter will be of the right consistency, the small batch will be excessively wet, causing segregation and its attendant evils. Although the cement per batch in each case is the same, the cement used per cubic yard of concrete is from 10 to 20% more in the one case than in the other, with no gain otherwise to compensate for the extra cement used.

An improved method is the use of some form of batcher. In this apparatus the method of filling the measuring box is standardized so that the same volume of aggregate is obtained each time. Even this does not eliminate variations in the absolute volume due to changes in grading and moisture content. These variations must be compensated for, when the error introduced warrants, but they seldom occur so rapidly as to cause any considerable variation from batch to batch, and their magnitude probably will not exceed 5%, except under unusual conditions, which can be provided for.

By inundation, a recent improved method, the fine aggregate is measured with its interstices filled with water. Under these conditions the absolute volume of the aggregate is the same as when measured dry. The scheme provides a standard method of filling and eliminates the variations due to moisture in the aggregates, but it does not eliminate the variations due to grading. Inundation is used as yet only for measuring the fine aggregate; the coarse aggregate still is measured by batchers and thus the method does not provide against the necessity of compensating for the water contained in the coarse aggregate, when determining the quantity of water for the batch.

Still another method that is being used successfully is weighing the aggregates. The weight of a given dry aggregate is a direct measure of its absolute volume as long as its specific gravity does not change. The method is independent of the manner of filling the container, of the grading of the aggregate, or of the volume changes caused by contained moisture, but the weight of the moisture must be allowed for if the absolute volumes desired are to be obtained exactly and for purposes of proportioning the water.

Moisture Content.—The only control that is possible where unstruck measurements are used is by proportioning the water and cement so that under the worst conditions concrete of the required strength and of a plastic consistency will be obtained. The more exact measurement of the other methods improves the uniformity of the concrete and thus makes close control possible. No method entirely eliminates the necessity for determining the

moisture content of the aggregates although inundation, by measuring the fine aggregate wet, solves this difficulty in part.

The apparatus for either of the volume methods must be calibrated to determine the relation between absolute volume and measured volume for the existing conditions of operation and whenever the gradation of the aggregate changes greatly, re-calibration becomes necessary. The measures in both are adjustable but changes in their capacity require considerable time and, in in an operating plant, are troublesome. Weighing plants offer certain advantages in flexibility as they can be changed from batch to batch without delaying operation; and also some advantages in accuracy since the measurements are not affected by changes in gradation, and but little by changes in moisture content. With any of these methods, uniform concrete can be produced and the advantages of one over the other are matters of convenience and economy.

As regards moisture in aggregates, the writer's experience has been that the errors in volumetric measurement caused by varying moisture content in the aggregates will ordinarily be less than 3% for the fine and 1% for the coarse, and are not, therefore, of great practical concern. They are in fact of less magnitude than the errors to be expected from the ordinary variations in gradation of the aggregates, which are usually entirely neglected; but the errors that variations in the moisture content of the aggregates will cause in the water-cement ratio, and hence in the quality of the concrete, are not negligible, for errors from this source will make about 3% difference in the gross quantity of water per batch for each 1% of uncompensated moisture in the fine aggregate, and twice this for a similar error in the coarse aggregate. This 3% difference may cause, in turn, a decrease in strength of 150 lb. per sq. in., or more, for concrete proportioned for 2 000 lb. per sq. in., at 28 days. The possible effect of uncompensated moisture may be seen by considering a change in moisture of, say, 4% from the normal water content of a well-graded concrete sand—a not unusual variation. This would cause a change of only 2 or 3% in the absolute volume measured by batchers, or similar means, but it would reduce the strength of the concrete 600 lb. per sq. in., or more, due to the effect on the water content of the mixtures, a reduction sufficient to change the character of the concrete entirely.

Manufacturing the Concrete

Mixing.—After measurement comes mixing. The minimum time of mixing recognized by any of the standard authorities is 1 min. after all materials are placed in the mixer drum. In practice, the tendency is to shorten this and even on major work the writer has observed mixing periods as short as 20 sec. A period of 1 min. is not too long; to mix less than 1 min. is to sacrifice some of the most desirable properties of concrete. Mixing is more than mechanically combining the ingredients; to be complete it must include a kneading and working so as to form a plastic, cohesive, and homogeneous mass that does not segregate in handling. Mechanical combination alone requires at least 30 sec. and it is only after this that the kneading action begins. Well-mixed concretes are stronger than under-mixed concretes, but

more important, they are more water-tight and durable—properties not to be despised and which many structures built in recent years apparently lack.

Placing.—From the standpoint of production, placing is a problem of transportation; but from the standpoint of quality and control, it is largely a problem of preventing segregation. No matter how scientifically the mixture has been designed, how accurately proportioned, or adequately mixed, it does not make good concrete unless it is uniform in quality throughout, and only careful workmanship in placing will bring this about; but workmanship can be assisted and segregation made less liable by properly designed mixtures and by a uniform batch thoroughly mixed.

Good workmanship in placing requires that concrete be deposited as near its final position as possible; that it be consolidated by puddling; that pockets of coarse aggregate be prevented both in the interior of the mass and at the exposed surfaces; that it be brought up evenly in the forms; and that the bonding of the concrete to adjacent surfaces, whether of rock or other concrete, be properly accomplished. Then there are certain "don'ts". The concrete should not be allowed to drop into the forms from too great a height; water should not be permitted to accumulate on top; corners should not fill with mortar and laitance; and the surfaces should not be spaded any more than is necessary for consolidation, for just as excess troweling ruins a horizontal surface, such as a road or floor, so excess spading is detrimental to a vertical surface. All these elements of good workmanship have the same object, to reduce segregation.

Why is segregation so objectionable? Segregation means patching and extra finishing; but it also means leaky structures when water-tight concrete is desired, zones of low strength (perhaps in dangerous locations), incomplete bonding of reinforcement, laitance seams, disintegrating joints (both vertical and horizontal), and honeycombed surfaces. Segregation cannot be corrected once the concrete is in place. Its effects can be hidden for a time but in concrete exposed to the weather, every patch, every place where mortar and coarse aggregate have separated to any degree, soon become plainly visible. There is no cure for segregation; if not wanted it must be prevented, and there is no prevention so effective as the maintenance of a high standard of workmanship.

Curing.—Lack of curing is just as effective in lowering the quality of concrete as lack of cement, excess water, or poor workmanship. Concrete requires to be properly cured if full advantage is to be taken of its inherent possibilities, and proper curing is more than merely a superficial attempt at preventing its surfaces from drying out. Many builders do not consider proper curing essential; others claim that it delays the work. Neither position is justified. Proper curing is essential and careful planning can produce it without detriment to progress.

All this discussion may seem foreign to the question in hand—the control of quality in the field—but control has been discussed so frequently in connection with the application of some special method of designing mixtures that its dependency on other factors, notably on the methods of measurement and high standards of workmanship, needs to be emphasized again and again.

Methods of Hydro-Electric Power Commission of Ontario

For controlling mixtures there are several practical means that have met with success, but the writer will outline only the methods of the Hydro-Electric Power Commission of Ontario. Since about 1920 the Commission has applied scientific control to the manufacture of about 640 000 cu. yd. of concrete on fifteen different operations, ranging in size from less than 1 000 to more than 500 000 cu. yd., some of it built by contract, and some by the Commission, so that the system of control developed is the result of extended and varied experience.

Preliminary Tests.—The initial steps—the preparation of specifications and the studies leading to the choice of aggregates—differ little from usual practice except that the specifications contain provisions that allow the engineer some latitude in setting his mixtures. They do not specify the strength although all concrete is so classified and they leave to the engineer the determination of the proportions that, in his judgment, will give the desired results.

Before concreting commences, proportioning tests are made in the laboratory with the cement and aggregates to be used on the job. This is done on all except the small jobs, less than 1 000 cu. yd. in size. This series of tests consists of five or more groups of five 6 by 12-in. cylinders proportioned to give at 28 days compressive strengths covering the classes of concrete used by the Commission. These test specimens are made in accordance with the standard requirements of the American Society for Testing Materials, except that the proper consistency is left to the judgment of an experienced operator, no reliance being placed on slump or flow tests, although the latter are usually made for purposes of record.

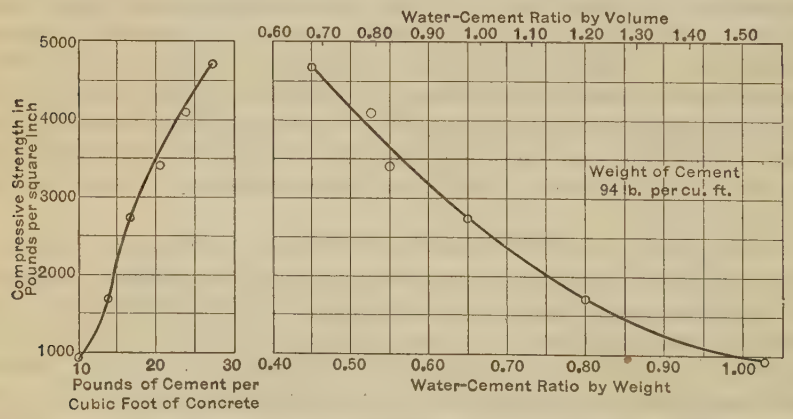
Water-Cement Ratio by Volume
0.60 0.70 0.80 0.90 1.00 1.10 1.20 1.30 1.40 1.50
5000
4000
3000
2000
1000
Compressive Strength in Pounds per square inch
Weight of Cement 94 lb. per cu. ft.
10 20 30
Pounds of Cement per Cubic Foot of Concrete
0.40 0.50 0.60 0.70 0.80 0.90 1.00
Water-Cement Ratio by Weight

Fig. 1.—Proportioning Tests, Job A, Hydro-Electric Power Commission of Ontario.

The proper ratio of fine to coarse aggregate for these tests is decided beforehand. The best ratio is largely a matter of judgment, based on the grading of the aggregates and a knowledge of the proportions that have given economical and workable mixtures with similar materials—a knowledge gained partly from laboratory studies and partly from field experience.

These tests establish the water-cement-ratio-strength relationship, the cement content for the materials, and the consistency and proportions; from these are set, for field use, the limiting quantities of water and the probable cement contents for concrete of the different classes. Such a group of tests are given with the field data derived from it in Fig. 1 and Tables 1 and 2.

TABLE 1.—Data Obtained from Proportioning Tests, Job A.

Ratio of fine aggregate to coarse aggregate: 40 : 60 by weight.
Fine aggregate: Graded 0 to No. 4; surface area, 20 sq. ft. per lb.; fineness modulus, 2.60.
Coarse aggregate: Graded No. 4 to 1½-in. round opening; surface area, 0.7 sq. ft. per lb.; fineness modulus, 7.14.

Serial No.	Cement, in percentage of aggregate.	Water, in percentage of aggregate.	Flow.	Concrete, weight per cubic foot.	Compressive strength, in pounds per square inch.	Water-cement-ratio by weight.	Cement, in pounds per cubic foot of concrete.
2 032	25	11.3	165	151	4 650	0.45	27.7
2 033	20	10.4	170	152	4 030	0.52	23.3
2 034	17	9.4	154	152	3 460	0.55	20.5
2 035	14	9.1	151	151	2 790	0.65	17.2
2 036	11	8.8	155	149	1 690	0.80	13.7
2 073	8	8.2	148	148	960	1.03	10.0

TABLE 2.—Proportioning Data, Job A.

(Use for ratio of fine aggregate to coarse aggregate : 40 : 60 by weight.)

Stated in Canadian Units.

Class of concrete.	Desired compressive strength at 28 days.	Cement, percentage of total aggregate.	Water-cement-ratio by weight.	Concrete, weight per cubic foot.	Gallons of water per bag of cement (Imperial gallons).	Bags of Cement (87.5 lb. gross) per Cubic Yard of Concrete.	
						In test.	Use.
A	2 500	13.0	0.68	151	5.8	5.2	6
B	2 000	11.5	0.75	150	6.4	4.6	5.25
C	1 500	10.5	0.85	149	7.3	3.8	4.25

Stated in United States Units.

Class of concrete.	Desired compressive strength at 28 days.	Cement, percentage of total aggregate.	Water-cement-ratio by volume.	Concrete, weight per cubic foot.	Gallons of water per bag of cement (U. S. gallons).	Bags of Cement (94 lb. net) per Cubic Yard of Concrete.	
						In test.	Use.
A	2 500	13.0	1.02	151	7.6	4.85	5.6
B	2 000	11.5	1.12	150	8.4	4.3	4.9
C	1 500	10.5	1.27	149	9.6	3.5	4.0

Choice and Verification of Mixtures.—From these data the field engineer determines the final proportions. He first makes tests to find the moisture

present in the aggregate and, if the materials are to be measured by volume, the weight of the aggregates per cubic foot under the operating conditions of the plant. Then from the size of the mixer on the job and the class of concrete required, he calculates in turn:*

(*a*) The quantity of cement per batch;
(*b*) The total quantity of water per batch;
(*c*) The quantity of fine and coarse aggregate per batch;
(*d*) The quantity of water contained in fine and coarse aggregates per batch; and
(*e*) The quantity of water to be added per batch.

This done, one or more full-sized batches are mixed and observed. If the consistency appears to be too dry, cement and water are added, keeping the percentage of water within the limits for the class in hand; if the batch appears to be too wet, cement and water are similarly deducted. If the mixture seems harsh, the fine aggregate is increased; if sandy, a quantity is left out. These changes may require a re-adjustment of the cement and water to restore the consistency. As it is ordinarily impracticable to measure cement in smaller units than a half bag, these changes may also require a re-adjustment in the size of the batch. In other words, the field proportions are finally determined by trial and error, to meet the two requirements of a limited quantity of water and a suitable workability.

From time to time these proportions may have to be altered to suit changes in the aggregates, but experience shows that the need for this is not nearly so great as would be expected. The writer has in mind a job of 10 000 cu. yd. for which the batch quantities of fine and coarse aggregate remained throughout at 1 320 and 2 100 lb., respectively. These weights were net and did not include the contained moisture, which varied slightly from day to day and was adjusted for as required. The engineers of the Hydro-Electric Power Commission have found that, generally speaking, there is no necessity either from the standpoint of control or economy, for redetermining the proportions for each new lot of aggregate. Any differences in their characteristics of sufficient consequence to warrant altering the proportions will manifest themselves in the consistency, which may be restored by changes in the quantity of cement and water, or less frequently, in the quantities or relationships of the aggregates.

Method of Control.—From this point, control takes the form of competent supervision of mixing, placing, and curing, and a periodic checking of the results obtained by tests on samples of the concrete produced.

This method differs in several important particulars from other methods of design and control and since these differences are not the result of chance, but are based on research and experience, it will be of interest to consider why they exist.

The method uses neither fineness modulus, surface area, or any similar factor for proportioning the mix. At the beginning, a modification of the surface-area method of Edwards was used for determining the proportions of

* For an example of detailed calculations, see Appendix.

cement, but this was found to be a needless complication and was abandoned during the third year. Considerable study has been given to different methods of designing mixtures, but invariably they have proved complicated, and, since satisfactory results are being consistently obtained without reference to such methods, there has been no reason for their adoption.

The second difference is that the water-cement-ratio-strength relationship is established for each job and for the materials and consistency to be used on that job. Except on small work, no reliance is placed on general curves, such as those developed by Abrams. The Commission's investigations have shown that the water-cement-ratio-strength relation is not the same for all conditions but differs for different cements and consistencies and for different aggregates and combinations of aggregates.

Abrams proposed two curves for designing concrete mixtures—one, the average of many groups of tests in which the proportions, materials, consistency, and grading of aggregates varied widely; the other based on this average curve but chosen to give greater strengths for the same value of water-cement ratio. Now, with any representative curve, the expectation is that one-half the tests of which it is the average will fall below it; consequently, in using such a curve to design concrete mixtures, one-half the mixtures designed could be expected to be lower in strength than the curve. This contingency would be covered by using a curve chosen to give strengths several hundred pounds per square inch greater than the average curve; but in order to be safe, such a designing curve sacrifices accuracy and becomes only an approximation of the real relationship. Further, it cannot be economical because, in the majority of cases, its use will give concrete of a higher quality than that desired. For these reasons, accurate and economical proportioning requires that the individual water-cement-ratio curve applicable to the conditions in hand be determined in each case.

A third difference is that no field test is used for consistency. The well-known slump test was early found to be inapplicable to lean mixtures and to crushed rock aggregates. As the slump test was then and is yet the only practicable field test available for consistency, it was necessary to develop a method of control independent of consistency tests. As yet no difficulty has arisen in the Commission's work to show that such a test is necessary. However, the writer would not quarrel with those who use the slump test for its widespread adoption has shown that it fills a definite need.

There is a fundamental difference between the viewpoint of the Commission and of those who use slump to control concrete mixtures. No matter what method is used, a concrete mixture of a given water-cement ratio and consistency is desired. If conditions are constant, changes in the water-cement ratio will be reflected in changes in the consistency and *vice versa.* However, there is no definite relation between the two and if conditions are not constant, the water-cement ratio may vary materially without appreciable variation in the consistency and hence in the slump. If the water-cement ratio is to be controlled with certainty, it should be controlled directly, and consistency, the control of which is not in itself of importance except as it

bears on problems of handling and placing the wet mixtures, can be maintained within a sufficiently narrow range for these purposes without testing. The experience of the Commission has proved such to be the case.

Field Testing.—The remaining step in the control of concrete as practiced by the Commission is the taking and testing of samples of job concrete as a record of performance and as a check on the work.

The Commission places a field laboratory on all except the smallest of jobs. This practice has been adopted because it has been found impossible to ship test cylinders by express with any assurance that curing will not be interfered with in transit. These field laboratories are equipped with a simple and inexpensive 100-ton hydraulic press, sieves for both fine and coarse aggregate, scales, pans, measures, moulds, heaters, etc. The total cost of this equipment in Canada (most of it has to be imported) is less than $1 000. Interest and depreciation are about $200 per job, but this cost is largely offset by the saving in express charges.

In proper hands these little laboratories are capable of surprisingly good work and make it entirely feasible to carry out the tests required to establish proportioning data such as is used by the Commission. They have a psychological value not to be overlooked, for it is possible for the resident engineer or inspector to demonstrate facts which no amount of data or argument would ever satisfactorily prove to the average superintendent or foreman.

Groups of cylinders are tested periodically throughout the concreting operations. These specimens are taken more frequently at the start of a job than after it is well organized and more frequently when difficulty with materials or control is being experienced than when the work is going smoothly. Each group consists of three specimens, one tested at 7 days, the others at 28 days. Whenever possible, the concrete from which the specimens are made is taken from the forms; when this cannot be done it is taken from the discharge of the mixer. Standard methods are followed in moulding, curing, and testing, particular care being taken in the matter of curing, for unless proper curing temperatures are maintained the results are not indicative of the quality of the concrete.

Interpretation of Results.—The results of tests on field samples are more than indications of the compressive strength of the concrete; they reflect the character of the job and the methods followed. Uniform test results usually indicate uniform methods of measurement. If in addition their average approaches in value the design strength of the concrete they offer the further information that the basis of design and the interpretation of the proportions into measured quantities of materials are correct. In the case of the method of design followed by the Commission, uniformly high results would usually be caused by wrong measurements. A check on this could be had by comparing the cement content of the concrete with that indicated as proper by the proportioning tests. When a method of design is used based on averages, high results similarly may indicate inaccuracies or that the design data used are not applicable, without correction, to the job in hand. A record of tests wherein the design strength is always exceeded but the results vary,

does not indicate necessarily a scientifically controlled job. If sufficient cement is used, excess water eliminated, and care exercised in sampling and testing, ample strength is almost inevitable.

There are bound to be certain groups of tests that depart considerably from the average. Oftentimes the unusual results are caused by faults in making the specimens or preparing them for test; the evidence of this can be found in the specimen. Occasionally no adequate explanation can be found. It is not the individual low or high test that is important, but a succession of low and high tests, or tests alternately high and low. Each of these conditions has special significance and the reasons therefor should be found and corrected wherever possible.

The part supervision plays in the successful production of a quality concrete should not be overlooked. Intimately connected with each job there should be a man who has a sufficient knowledge of the properties of concrete to know the effect of the different malpractices to which it is subjected during manufacture, and who knows of what good workmanship consists and how to get it. Unless the concreting operations are in the hands of trained and competent men, the results of attempted control are almost certain to be disappointing.

Cost.—Supervision such as is required in control work cannot be had for nothing, but through it many economies are possible. For example, on one large project the designing engineers used concrete of a lower grade as soon as it was demonstrated in the field that concrete of a specified compressive strength could be consistently produced, and this resulted in a considerable saving. On another job the use of accurate measuring equipment not only resulted in considerable saving in cement due to closer proportioning, but also lowered concreting costs by increasing the output of the mixing plant, by reducing the delays in handling, and by making control easier.

In general, the cost of scientific control is little more than for the type of supervision usually employed and the economies that result when a concerted effort is made to produce a uniform, well-placed concrete will actually result in a cheaper concrete. That it is profitable is proved by the fact that few who try control methods later abandon them and that, as time goes on, they are being applied to smaller and smaller jobs.

As there is no royal road to fortune, there is likewise no easy way of making good concrete. Control will not be attained by following any particular theory as to the selection of materials, method of proportioning, etc., but is always the product of forethought and care in respect to every detail of its manufacture. If there is a formula for success, it is as follows:

Controlled concrete = suitable materials + a bit of theory applied with common sense + good workmanship + competent supervision — nothing more and nothing less.

APPENDIX

The following case is given in detail to illustrate the method followed by the Hydro-Electric Power Commission of Ontario in the calculation of the field proportions of concrete.

The conditions assumed are as follows:

Information desired.	Source.	Data obtained.
Capacity of mixer	Job plant	1 cu. yd.
Class of concrete	Engineering drawings	*Class A
Moisture in sand	Field tests	3.4% by weight
Moisture in gravel	Field tests	0.5% by weight
Weight of concrete	Proportioning data, Table 2.	151 lb. per cu. ft.
Ratio of fine to coarse aggregate	Proportioning data, Table 2.	40 : 60 by weight
Cement per cubic yard.	Proportioning data, Table 2.	6 bags
Water-cement ratio	Proportioning data, Table 2.	0.68 by weight

From the capacity of the mixer can be determined the weight of a wet batch of mixed concrete:

Capacity of mixer.....27 cu. ft. of wet concrete
Weight of batch.....$27 \times 151 = 4\,080$ lb.

This 4 080 lb. is made up of:

6 bags of cement..... $6 \times 86\dagger = 516 = 520$ lb.
Water ..... $520 \times 0.68 = 354 = 354$ lb.
Aggregate .....$4\,080 - (520 + 350) = 3\,210$ lb.

Of this, the aggregate is divided as follows:

40% is sand.....$0.4 \times 3\,210 = 1\,280$ lb.
60% is gravel.....$0.6 \times 3\,210 = 1\,930$ lb.

The weight of moisture must be determined and corrected for:

Weight of water in sand.....$3.4 \times 1\,280 = 44$ lb.
Weight of water in gravel.....$0.5 \times 1\,930 = 10$ lb.

The field weights are, therefore:

Sand .....$1\,280 + 40 = 1\,310$ lb.
Gravel .....$1\,930 + 10 = 1\,940$ lb.
Water ..... $354 - 54 = 300$ lb.

Hence, the trial proportions if by weight are:

Cement ..... 6 bags
Sand (moist) .....1 310 lb.
Gravel (moist) .....1 940 lb.
Water ..... 30 gal. (Imp.), or 36 gal. (U. S.)

or by volume, the trial proportions are (assuming the weight per cubic foot of the materials measured at the mixer as being 110 lb. for gravel and 80 lb. for sand):

Cement ..... 6 bags
Sand (moist) $1\,310 \div 80$ .....16.4 cu. ft.
Gravel (moist) $1\,940 \div 110$....17.6 cu. ft.
Water .....30.0 gal. (Imp.), or 36 gal. (U. S.)

* See Table 2 for specification of classes used by the Hydro-Electric Power Commission of Ontario.

† The Canadian bag is 87½ lb. gross. The weight, 86 lb., is net, based on an allowance of ½ lb. for the sack and 1 lb. for cement left in the sack after emptying into charging hopper.

DISCUSSION

T. P. WATSON,* M. AM. SOC. C. E.—Engineers and correlative interests are greatly indebted to Mr. Young for the presentation of this comprehensive paper on a much mooted subject. The pioneering of Mr. Young and his Canadian associates in the large-scale production of "uniform quality concrete" is worthy of unstinted praise and is an excellent example of what can be accomplished.

A study of the methods explained in the paper, and their comparison with general practice, impresses those who have been familiar with concrete production during the past twenty years, that the art of manufacturing concrete has not kept pace with the use of this valuable construction material, and that the transition from "placing" concrete to "pouring" it passed without sufficient attention being given to the use of excessive quantities of mixing water.

The use of excessive quantities of water, to take advantage of mechanical and other means to decrease labor costs and to meet the urge of the maximum possible production, continued until the actual failure of concrete structures awakened the dormant allied interests to the seriousness of the situation.

The views being put forth for the production of better quality and uniform concrete mixtures are usually not new, but are rather a revival and application to present-day requirements of the theories presented prior to 1900 by various investigators whose warnings were passed unheeded or forgotten.

The water-cement-ratio theory of strength was proven by tests† thirty years ago, and the theory of proper proportions—that the voids in the coarse aggregates should always be filled with mortar—was presented about the same time. Practically every book on the subject of concrete has called attention to the laxity of using arbitrary loose proportions of aggregates regardless of their sizes and shapes.

In spite of the discussion of this subject and the publicity given to the active demonstration of Mr. Young and his associates, as well as many other reputable concrete users, that the accurate control of concrete manufacture is both practical and economical, the majority of concrete operations are continuing in the use of haphazard and careless methods, disregarding water content and a rational proportioning of aggregates, even although casual observation will prove that many of these structures exposed to northern climatic conditions are almost certain to decay in a comparatively short time.

A glaring reflection on the Engineering Profession, as regards concrete manufacture, is the apparent lack of comprehension of the function of competent field supervision. As long as engineers fail to realize that adequate and competent field supervision and inspection are necessary to control the handling of the product from the time of proportioning the aggregates until the concrete is in place and cured, the problem of producing "uniform quality concrete" will not be solved.

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* Prin. Asst. Engr., Philadelphia Impvts., P. R. R., Philadelphia, Pa.

† Report of the New York State Engr. and Surv., the late George W. Rafter, M. Am. Soc. C. E., 1897, pp. 375–460.

Engineers can advantageously ponder Mr. Young's admirable concluding formula:

"Controlled concrete = suitable materials + a bit of theory applied with common sense + good workmanship + competent supervision—nothing more and nothing less."

HARRY C. BOYDEN,* M. AM. SOC. C. E.—The author has stated that scientific control costs money. Is it not true, however, that proper supervision placed on the work—without any attempt to enter into scientific control—would have cost exactly the same? In other words, the institution of scientific methods on the hydro-electric work did not entail added cost, above that of proper supervision for any good job.

Another question: In placing 650 000 cu. yd. of concrete on these projects what has been the tangible and intangible saving in cement beyond the amount that would have been used by following ordinary methods? Apparently, savings have been effected on that work running into hundreds of thousands of dollars, but the author is altogether too modest to claim the credit. This saving was due to the methods of control used. These methods did not entail additional expense and, more important than the saving in dollars, they obtained the quality of concrete called for in the design of the structures.

WILLIAM H. ADAMS,† M. AM. SOC. C. E.—The author has discussed a type of work which comes under strong centralized control. It would be interesting to hear discussion of possible control of quality of concrete on a more commercial job.

In the ordinary commercial building operation the structural plans are frequently made for the architect by a consulting structural engineer. The inspection and supervision of all work, including structural, in the majority of such cases, are under the control of the architect's superintendent who is rarely an engineer, although usually well versed in building construction.

On by no means all such operations is the cement tested by an independent laboratory, although such test requirements are usual in the most important work. Has a point been reached where the control of the physical quality of aggregates can be delegated to a standard commercial testing laboratory? Are laboratories equipped for this service?

With such marked improvement in strength of resultant concrete as Mr. Young's paper indicates will follow from the use of carefully controlled water-cement ratios, it should not be difficult to make an owner understand that efficient laboratory inspection would be an investment and not an expense.

THOMAS K. A. HENDRICK,‡ ASSOC. M. AM. SOC. C. E.—It is the understanding of the speaker that the Hydro-Electric Power Commission of Ontario has used, during the course of the installation of about 640 000 cu. yd. of concrete under scientific control, coarse aggregate of every kind, including gravel and broken stone.

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‡ Civ. Engr., New Rochelle, N. Y.

Under the general heading of "Selection of Materials", Mr. Young states that,

"* * * although control can be obtained when the materials vary, it is obtained more easily and cheaply when they are uniform. For this reason it is desirable to limit the supply of each to one source and to choose those known to vary least, even when this involves some additional expense."

In the use of a local stone for the coarse aggregate, which is apparently of poorer stuff, the expense of transportation of the apparently better coarse aggregate is overcome. Since in many cases to secure the required strength it would be necessary to use more cement, of course the use of the local material is not always a clear economic gain even if transportation expense is avoided.

Experiments performed by the writer* showed that, in some instances, as in the case of the Minnesota Kettle River sandstone which pulverizes easily under the pressure of the thumb and forefinger, a concrete may result which is higher in strength than that made with trap-rock, gravel, or granite. The same general conclusion has been reached independently.†

It has not been demonstrated that the densest concrete gives the highest strength. Repeated experience has shown it to be highly desirable to experiment on each local material that has any potential suitability as a coarse aggregate. If the author would limit the supply to one source, it should be stipulated that many trial comparative experiments be performed on every available local stone before a decision to limit to the one source be reached. To secure uniformity is the desideratum; but uniformity can be secured by combination of methods rather than at the sacrifice of a local material without fair trial.

John G. Ahlers,‡ M. Am. Soc. C. E. (by letter).—The wealth of experience shown in Mr. Young's paper is very impressive and adds greatly to the conviction of his conclusions.

As he states the entire problem of producing concrete of uniform quality is really an economic one and resolves itself into, first, arriving at the proper water-cement ratio for any required strength; and, second, combining the materials available so as to give the highest yield with the selected water-cement ratio.

Like many others, Mr. Young forgets that strength can be used as a criterion of all the other desirable qualities of concrete. Contrary to his opinion, the writer believes that the design or mixture should be based on strength alone. If a high strength is obtained, all the other desirable qualities will follow—as proven by Professor Abrams§ and as shown by Professor M. O. Withey, of the University of Wisconsin, in connection with permeability. If plasticity and workability are made the limiting factors then

* "A Study and Determination of the Relative Efficiency of Sixteen Different Kinds of Broken Stone and Two Different Kinds of Gravel When Used as Aggregate in Concrete," *Engineering News*, March 9, 1916.

† "An Investigation of the Concrete Road Making Properties of Minnesota Stone and Gravel," by Prof. Charles Franklin Shoop, *Studies No. 2*, Univ. of Minnesota, 1915.

‡ Secy.-Treas., Barney-Ahlers Constr. Corporation, New York, N. Y.

§ "Bond Between Steel and Concrete."

the measurement, fineness modulus, and volume of fine to coarse do not require exact control, and the whole problem of producing concrete of uniform quality becomes very simple.

It is particularly interesting to note that in the large volume of concrete produced under Mr. Young's supervision, he finds that the method of handling material prior to measurement is not of great effect on the quality of the concrete, except that any method which allows segregation of the aggregate makes it more difficult to obtain uniform concrete. The writer's experience is that concrete of uniform strength is produced even if there may be segregation, but the yield or economy of the different batches may vary. In other words, in a poorly graded batch not as large a volume of concrete will be produced as in a batch with the fine and coarse aggregate well graded.

The writer rather questions the statement that the measurement of cement is not a problem of control. It is a known fact that the different brands of cement will have different weights in the bags and that within the same brand there will be variations from bag to bag. Also, there are many bags broken on every concrete operation, and many bags with lumps. It is not often that the bags are shaken in the same manner each time. It, therefore, becomes very important to watch the quantity of cement going into every batch. A system that allows simple supervision of this quantity will simplify the problem of control. A system that uses the cement as a measure of the water makes this control absolute and allows no variation regardless of the irregularities in the measurement of cement.

Regarding the moisture content Mr. Young's experience parallels that of the writer to the effect that no method entirely limits the necessity of determining the moisture contents of the aggregate. It is extremely important that moisture be determined in both coarse and fine aggregate and allowance made for the average maximum condition. By so doing the concrete will be slightly stronger than is expected or required.

As to the importance of the period of mixing, if the testing of the concrete produced under job conditions is made the standard from which other strengths may be obtained, the period of mixing does not enter into the problem of field control. It is a fact, however, that contractors may obtain higher strengths and, therefore, concrete at lower cost if they allow a longer mixing time when producing concrete for a given strength.

The paper is very helpful in further developing the general knowledge of field control of concrete. The system of field control is becoming more standardized and simplified each year. On field work undertaken by the writer's organization it has reduced to the following simple rules:

(1) Under job conditions manufacture concrete with a fixed water-cement ratio. This is called the "job constant of strength" and fixes the "job curve".*
(2) Test this concrete at 7 days and interpret the test results for 28 days.
(3) From these test results determine what other water-cement ratios will give the strengths required on the job.

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 823, *et seq.*

(4) Use a mechanical device for giving these water-cement ratios and add as much as possible well-graded fine and coarse aggregate to the water-cement mixture.
(5) Check the resulting concrete from time to time against the "job constant of strength".

William M. Hall,* M. Am. Soc. C. E. (by letter).—Uniform quality in the production of concrete is a subject that has probably received as much attention by engineers during the past few years as any other presented before the Society. It has been under discussion with more or less activity since the commencement, about 1892, of monolithic concrete construction on the Hennepin Canal, where Col. W. L. Marshall, U. S. A., used concrete so dry as to give no slump, and on the Chicago Canal where the late Isham Randolph, M. Am. Soc. C. E., Chief Engineer, used concrete wet enough to give a slump of 9 or 10 in. Hence, it hardly appears creditable to the profession that after such a long time it should be subject to such a general indictment as given† by George F. Swain, Past-President, Am. Soc. C. E. However, it is notorious that a considerable percentage of the great volume of concrete constructed in the United States is believed to be justly subject to such criticism.

The author stresses the fact that for uniformity it is essential to have unvarying absolute quantities of the several ingredients, including water; that such a mixture would be uniform also in strength and easy of control.

Of the highly desired qualities of concrete in most concrete structures the one quality in which uniformity is most desirable is its strength. As stated by the author, a uniform mix is the basis of uniform concrete; but it is the basis only. The lack of uniformity in the materials is a basic condition preceding the mix, which affects the uniformity of the concrete. There is a lack of uniformity in the strength of the cement, in which it is not unusual to find a variation from 20% to as high as 50% in 7-day briquettes and nearly as great a variation in the 28-day briquettes. Such variations are found in many brands of cement.

With other conditions uniform, such variations in the strength of the cement will produce a corresponding variation in the resulting concrete. If the percentage of resulting variation in the strength of concrete is not as great as that in the briquettes made from the same cement, it will probably be close to it. Therefore, it appears most desirable to cure that defect in cement. Until it is cured, it seems impracticable to prevent the variation in the strength of concrete produced therefrom.

Another condition affecting uniformity of strength is the great variation in both the fine and the coarse aggregate, in such localities as the sand and gravel bars in the Ohio River between Pittsburgh, Pa., at its head, and Cairo, Ill., at its mouth, 967 miles. These bars are producing material in large quantities, and will probably continue such production for centuries. It is believed that this variation in the grading of gravel at many such localities, all other conditions being equal, will produce as much as 5 to 20% variation in the strength of the concrete. Although this variation in grading

* Parkersburg, W. Va.
† *Engineering News-Record*, December 23, 1926.

is objectionable, the advisability of incurring the expense of correcting it by artificial re-grading is questionable. It is probable that more than 90% of the concrete made within 50 miles of the Ohio River is of gravel and sand from the river, and no one has attempted to re-grade it.

Possibly the condition of next importance in producing uniformity in concrete (and possibly of even greater importance than the two others named) is the lack of intelligent, well-trained, and enthusiastic workmen at the mixer and in the forms, who are actually executing all the details of keeping check on the proportioning, measuring, mixing, placing, and curing described by the author. Although more care is now being given to this training than a few years ago, it appears that even to approximate uniform quality of concrete all the men engaged in doing the work should be trained in the details of the methods of control relating to their respective jobs.

The author states that, "It is not the individual low or high test that is important* * *". The writer does not agree with this conclusion as to low tests, but thinks, on the contrary, that a single batch of concrete which averages below the minimum strength requirement is objectionable. Such deficiency is, of course, much more objectionable under some conditions, or in some members, than in others. It appears that the test, or series of tests, should indicate that any section through any member of a concrete structure of any kind has an average strength per square inch equal to the minimum strength for which it is designed. Until such degree of perfection is reached or closely approximated, it is probable that concrete as a building material will be subject to criticism.

The writer believes that the day is near when a minimum strength for concrete can be specified and obtained with less than 1% of the batches falling below the minimum, and with an average strength of all the concrete in any monolith, or in any two consecutive batches, above the minimum. After the minimum strength is attained or exceeded by all the batches, uniformity as to the variation between the highest and lowest strength batches is not of vital importance to the safety of the structure, and is of minor importance to the owner. However, the effort to obtain uniformity in strength is desirable and is a goal which should be sought by the manufacturers of cement as well as by the makers of concrete.

Roderick B. Young,* Esq. (by letter).—The writer is much gratified with the interest taken in this paper and feels well repaid for the labor of its preparation. The resulting discussion has been valuable and brings out a number of interesting points.

The questions raised by Messrs. Boyden and Adams are very pertinent. The writer has not found, in connection with those jobs with which he has been associated, that any increased help was required to apply the methods of concreting he has described, more than would have been required in any case to supervise the work properly. This has been true even where field laboratories were used, except possibly where the work was of such size that

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the regular staff would be fully occupied by the usual inspection duties, when assistance would be required to handle the testing and clerical work.

It is almost impossible to set a cash value on the benefits of scientific control. It is equally difficult to state with certainty how much cement was saved in any given case. In some of the earlier work carried out by the Hydro-Electric Power Commission of Ontario, it is doubtful whether any cement was saved. In every case, however, the quality of the concrete was improved; and if the cement used was compared with the quantity required to obtain equal concrete under the methods previously in force, the savings would be substantial. On the more recent work of the Commission not only has the quality of the concrete been bettered, but the cement used has been reduced from 15 to 25 per cent. Other savings besides a reduction in cement costs can be credited, at least in part, to the increased attention given to the methods of concreting. Probably the Commission has benefited by the amount of $1 per cu. yd. at least, on all concrete placed since the principle of scientific control was adopted.

The writer sees no insuperable difficulties in applying the methods he has described, or modifications of them, to almost any work where the will exists to make the effort. The Commission is using the same principles with success on small isolated jobs where supervision is necessarily limited. In many localities commercial laboratories are available, which can make the required tests; if not, a field laboratory can be installed at small expense. The great difficulty is to get men with the necessary faith, knowledge, personality, and experience to carry on the details and get the co-operation of the field forces.

The writer does not quite follow Mr. Hendrick in his discussion since the paper did not raise the question of local *versus* imported aggregates, but only stated that if uniform concrete was the desideratum it was of advantage to have the cement, fine aggregate, and coarse aggregate, each from a separate source. Surely it is not Mr. Hendrick's intention to imply that he considers it satisfactory to use material from one source to-day and from another to-morrow; but if it is, the writer can only reply that in his experience such practice is undesirable. It is seldom necessary and always increases the difficulty of obtaining a uniform quality of concrete.

Mr. Hendrick suggests exhaustive tests on all available aggregates before a decision to limit the supply to one source be reached. He is right in principle, but in practice the amount of testing that is justified in any given case depends on the size and importance of the work—a small yardage of concrete does not justify the expense of an elaborate series of experiments. The choice of an aggregate should be made on a basis of its ultimate cost in the concrete, of uniformity of quality, and of continuity of supply. It is a distinct economic asset to have aggregates that are of uniform quality because they permit not only that the mixtures may be designed more closely, but that the quality of the concrete may be controlled more cheaply.

Mr. Ahlers thinks that strength can be used as a criterion of all the other desirable qualities of concrete. Apparently, in common with many others, he holds to a belief that, paradoxically speaking, is true and yet not true. As a

broad general fact, it is true that the other properties of concrete are related to its strength, but the relationship is not fixed and, therefore, different concretes of like strength may have other properties, such as permeability, resistance to weathering, etc., in quite different degrees. For example, a concrete showing a strength of 2 500 lb. per sq. in. at 28 days will ordinarily be non-absorptive and highly resistant to frost action; but the writer knows of cases where concrete of this strength was entirely deficient in these qualities and failed in service, in one case disintegration being pronounced in less than 2 years. The design of a particular concrete mixture is concerned not so much with broad general truths as with a particular case where the exception that proves the rule may apply; it is necessary, therefore, in designing concrete for a certain purpose to give thought to the other properties required for that purpose and to modify the design accordingly.

Mr. Ahlers states that the length of the mixing period does not enter into the problem of field control of concrete. Of course, if one concedes that the strength of concrete is the only property that needs to be considered, then the mixing period becomes strictly an economic problem; but, if the contrary is true, the period should be regulated. As stated in the paper, increasing the time that the concrete is undergoing mixing imparts increased workability to the mixture and to the finished concrete, also durability and impermeability—properties of such importance, especially in an outdoor structure, that it would not be wise to ignore the benefits resulting from adequate mixing.

Mr. Hall's comments are well taken. There are variations in the quality of the constituent materials of concrete which, in turn, affect the quality of the concrete made from them. Differences in the cement, particularly, would fundamentally affect the quality of the concrete as measured by strength. It is doubtful, however, whether the differences recorded by the usual cement tests are an accurate indication of the differences that will result when the cement is made into concrete; whether variations in the grading of the aggregate need cause any considerable changes in the strength of the concrete made from it.

Mr. Hall refers to the importance of workmanship in the production of a uniform concrete. It is to be regretted that more engineers do not appreciate this fact and do not bend some of their efforts to giving the workman a better insight into the fundamentals of good concrete and into some of the evils resulting from practices common to-day.

Mr. Hall takes issue with the partial statement that, "It is not the individual low or high test that is important * * *." This is but part of a sentence and cannot be properly removed from its context. The writer was discussing the interpretation of test results, and his statement was to the effect that any individual high or low test would not singly be a reliable indication of markedly inferior or superior concrete because such abnormal results might be accidental, due to faulty test specimens or to errors of test, but that a succession of low or high results would be very significant and would indicate rather definite important changes in the quality of the concrete.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1632

WATER-RATIO SPECIFICATION FOR CONCRETE*

By F. R. McMillan,† M. Am. Soc. C. E., and Stanton Walker,‡ Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. C. R. F. Coutlee, A. S. Woodle, Jr., T. P. Watson, Charles S. Whitney, Harry C. Boyden, L. A. Perry, John G. Ahlers, Thaddeus Merriman, and F. R. McMillan and Stanton Walker.

Introduction

The control of quality of concrete in the field has received considerable attention in the technical discussions of recent years. There has been a growing demand that the results of the extensive laboratory and field investigations into the properties of concrete be made the basis of practice. The need for a specification that will meet this demand and at the same time form an equitable basis for a contract has been quite generally expressed.

It is the purpose of this paper to present a specification, which, the writers believe, fulfills these requirements. It is based on the fundamental principle that the strength and other desirable properties of concrete are definitely determined by the proportion of the water to the cement in the mixture, provided only that the concrete is plastic and workable and the aggregates are clean, durable, and structurally sound. It has been clearly established that the size and grading of the aggregate, the quantity of the cement, and the consistency of the concrete affect the strength only in so far as they influence the quantity of mixing water required to produce concrete of the necessary workability. Any combination of the constituent materials that provides a given water-ratio will result in concrete of the same strength as long as it is workable.

* Presented at the meeting of the Structural Division, Montreal, Que., Canada, October 15, 1925.

† Mgr., Structural and Technical Bureau, Portland Cement Assoc., Chicago, Ill.

‡ Director, Eng. and Research Div., National Sand and Gravel Assoc., Washington, D. C.

The importance of proper control of the water has long been recognized, but the definite law between strength and quantity of mixing water was developed* by D. A. Abrams, M. Am. Soc. C. E. It has been fully confirmed by later tests carried out by him and by many other investigators.

The significance of the part played by the mixing water is more readily appreciated when the cement paste is thought of as a glue binding the aggregates together. Too much mixing water dilutes the glue and hence reduces its strength and density.

Basic Data

Fig. 1 shows the relation between the compressive strength of concrete and the "water-ratio", that is, the ratio of the volume of water to the volume of cement (assuming 1 cu. ft. of cement to weigh 94 lb.), based on about 6 000 compression tests of 6 by 12-in. cylinders at the age of 28 days. The tests include mixes varying from neat cement to proportions of 1 cement to 15 aggregate, consistencies ranging from stiff to sloppy, and aggregates varying widely in size. From the legend on Fig. 1 it will be seen that, for the same water-ratio, the different mixes gave practically the same strength. This was true regardless of the size or grading of the aggregate or the consistency of the mix, except for the consistencies that were too dry for proper workability, which have been omitted from the diagram.

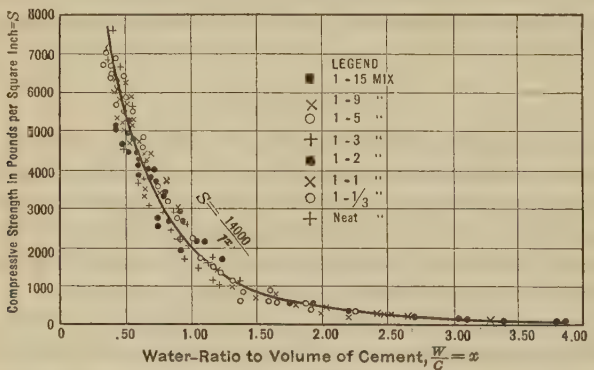


Fig. 1.—Relation Between Strength of Concrete and Water Content.

The water-ratio-strength relation in Fig. 1 may be represented by the equation:

$$S = \frac{14\,000}{7^x}$$

in which,

S = the compressive strength at 28 days, in pounds per square inch.; and
x (an exponent) = water-ratio (by volume).

Many subsequent tests have shown practically the same mathematical relation. Others give a similar curve, but with different constants in the equation.

* "Design of Concrete Mixtures," *Bulletin 1*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

Table 1 summarizes data from a recent series of tests carried out at the Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill., and demonstrates in a convincing manner that the strength of concrete is determined by the water content. It will be noted that for a given water-ratio (expressed as U. S. gallons per sack), the values are in almost perfect agreement, regardless of the mixture, while for a given mix, the strengths will be seen to fall off rapidly as the water-ratio is increased. These results show somewhat higher compressive strengths for a given water-ratio than those obtained from any previous series.

TABLE 1.—Compressive Strengths of Concrete for Various Water-Ratios.

Specimens: 6 by 12-in. cylinders.
Aggregate from Elgin, Ill.: Sand graded from 0 to No. 4 and pebbles graded from No. 4 to 1½ in.
Cement: A mixture of four brands of Portland cement purchased in Chicago, Ill.
Specimens cured in moist room at 70° Fahr. until test; tested damp.
Age at test: 28 days.
Each value is in pounds per square inch and is the average of five tests made on different days.

Mix by volume.*	Quantities of Mixing Water (U. S. Gallons) per Sack (94 Pounds) of Cement.								
	5.	5½.	6.	6½.	7.	7½.	8.	8½.	9.
1 : 1 : 1	5 250	4 520	3 960	.....	.....	.....	.....	.....	.....
1 : 1 : 2	5 420	4 680	4 080	3 450	.....	.....	.....	.....	.....
1 : 1 : 3	5 400	4 710	3 930	3 140	.....	.....	.....	.....	.....
1 : 1 : 4	.....	.....	3 810	2 800	.....	.....	.....	.....	.....
1 : 1½ : 0	5 380	5 070	4 300	3 970	.....	.....	.....	.....	.....
1 : 1½ : 1	5 170	.....	.....	.....	.....	.....	.....	.....	.....
1 : 1½ : 2	5 210	4 760	4 370	3 770	3 420	2 920	.....	.....	.....
1 : 1½ : 3	.....	4 780	4 650	3 990	3 540	2 620	2 380	.....	.....
1 : 1½ : 4	.....	.....	4 370	3 870	3 370	2 590	2 040	.....	.....
1 : 2 : 0	.....	4 150	4 090	3 740	3 340	.....	.....	.....	.....
1 : 2 : 1	.....	4 340	4 010	3 660	.....	.....	.....	.....	.....
1 : 2 : 2	.....	.....	4 320	3 640	3 160	3 060	.....	.....	.....
1 : 2 : 3	.....	.....	3 670	3 910	3 490	3 140	2 760	2 500	.....
1 : 2 : 4	.....	.....	.....	3 630	3 520	3 210	2 820	2 380	1 960
1 : 2 : 5	.....	.....	.....	.....	.....	2 750	2 370	2 170	1 920
1 : 2 : 6	.....	.....	.....	.....	.....	2 650	2 400	2 250	1 870
1 : 2½ : 0	.....	.....	3 560	3 510	3 430	3 210	.....	.....	.....
1 : 2½ : 1	.....	.....	3 590	.....	3 470	.....	.....	.....	.....
1 : 2½ : 2	.....	.....	.....	3 480	3 310	3 350	.....	.....	.....
1 : 2½ : 3	.....	.....	.....	3 360	3 100	3 090	2 950	.....	.....
1 : 2½ : 4	.....	.....	.....	.....	.....	2 950	2 730	2 790	2 460
1 : 2½ : 5	.....	.....	.....	.....	.....	2 740	2 440	2 410	2 240
1 : 2½ : 6	.....	.....	.....	.....	.....	.....	2 360	2 280	2 040
1 : 3 : 0	.....	.....	.....	.....	.....	2 980	2 710	.....	.....
1 : 3 : 3	.....	.....	.....	.....	.....	1 740	2 510	.....	.....
1 : 3 : 4	.....	.....	.....	.....	.....	2 580	2 460	2 320	.....
1 : 3 : 5	.....	.....	.....	.....	.....	2 320	2 460	2 300	2 100
1 : 3 : 6	.....	.....	.....	.....	.....	.....	2 320	2 230	2 020
Average	5 300	4 630	4 050	3 590	3 380	2 820	2 510	2 360	2 080

* In terms of dry and rodded volumes of aggregates.

Other desirable properties are usually obtained with concrete of a high compressive strength. Professor Abrams has shown definite relations between compressive strength and flexural strength, resistance to wear, bond resistance, etc. Unpublished data of the Structural Materials Research Laboratory show

that in general the resistance of concrete to destructive agents, such as sulfate waters, etc., increases with the compressive strength.

Withey states* that:

"For properly placed and cured broken stone or gravel concrete the permeability decreased as the compressive strength increased until the latter reached 2 500 lb. per sq. in. Concretes of this strength were water-tight."

He emphasizes also that it is essential that the mix be plastic.

Recent Suggested Forms of Specifications

Beside the water-ratio specification for quality of concrete, certain other forms of specifications have been advocated within the past few years which aim to insure the proper quality. The more usual form of specifications makes no mention of the quantity of mixing water and, therefore, does not definitely fix the quality. One of the recent forms is the specification of definite proportions of cement and aggregate together with the consistency of the concrete in terms of the slump as proposed by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its report of 1924.† The other is the specification of the strength of concrete as proposed by the Joint Committee in its Progress Report of 1921.‡

In the specification based on the proportions and slump, it is presumed that the engineer will investigate the available aggregates and make the necessary strength tests so that he can select combinations which, when mixed to the specified slumps, will give concrete of the desired strengths and workability.

Two major objections may be raised to this form of specification: First, it presents a fruitful source of argument—the workability. Although the slump test is useful in maintaining a constant quantity of water providing no important fluctuations in the grading or quantity of materials occur, it is not always a suitable measure of that intangible quality which may be called the workability, as, for example, with a well-sanded mix, a 2-in. slump may be much more workable than a "harsh" mix with a 6-in. slump. Even if, in the opinion of the engineer, the slump and mix are entirely satisfactory, there is still that tendency on the part of the workmen to increase the fluidity of the mix wherever possible. The second objection is that in his study of available aggregates, the engineer should give full consideration to the relative economy of different proportions in order that the mixes specified may not add an unnecessary financial burden to the structure. Rarely does the engineer have available the necessary equipment or the time to do justice to such an investigation.

The strength specification proposed in the 1921 Report of the Joint Committee as an alternative,‡ but not included in the 1924 report, reads:

"28.—*Proportions.*—The contractor shall use materials, so proportioned and mixed, as to produce concrete of the required workability and strength. Frequent compression tests of the concrete used in the work will be made by

* *Bulletin 1245*, Eng. Experiment Station, Univ. of Wisconsin, 1923.

† *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, pp. 1173–1174.

‡ *Loc. cit.*, August, 1921, Papers and Discussions, p. 74.

the Engineer, and in case of failure to meet the specified strength, the contractor shall make such changes in the materials, proportions, or mixing, as may be necessary to secure concrete of the required strength. * * *"

To many, this is the logical specification for concrete. A study of the unfavorable comments regarding this clause* shows that, in the present state of the art, most builders are not ready for a specification under which the contractor guarantees the strength. The principal objections raised were the following:

(1) It cannot be known until the concrete is in place and thoroughly hardened, whether it has met the strength requirements;
(2) Test methods are not so thoroughly developed or reliable as to eliminate entirely misinterpretation or abuse;
(3) No proper basis has been suggested for adjustment or penalties in case the strength fails to meet the requirements; and
(4) The engineer evades responsibility for the quality of the concrete by passing the burden to the contractor.

The advocates of the strength specification point out that through it they are offering to the contractor an opportunity to profit by the selection and proper use of good materials, and at the same time are insuring to the owner a structure of the required strength.

Water-Ratio Specification for Concrete

In the Appendix, there is given a complete specification based on the water-ratio, which, in the opinion of the writers, is free from most of the objections raised to the other forms mentioned and presents certain advantages not possessed by them. This specification has been used for a reinforced concrete building and some of the results achieved are referred to subsequently. The outstanding features of the specification are briefly these:

(*a*).—A definite water-ratio, corresponding to the strength desired, is specified for each grade of concrete. These water-ratios are the maximum permissible and include the moisture in the aggregate. The operating tolerance covering unavoidable fluctuations must be entirely on the lower side so that the ratios specified are not exceeded.

(*b*).—The proportions of aggregate to cement are governed only by the requirements of workability, with the proviso that the coarse aggregate shall not be less in quantity than the fine, nor more than twice the fine. Workability is limited by the requirements that the mass can be puddled readily into the form without harshness or honeycombing and without the accumulation of water on the surface.

(*c*).—The contractor is required to measure the materials in such a way that the water-ratio can be checked at any time, thus avoiding haphazard changes in consistency. This involves, of course, reasonably accurate measurements of the aggregate as well as a fair degree of uniformity in the materials as received.

* See *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, pp. 634–674; also Discussion of Joint Committee Report, *Proceedings*, Am. Concrete Inst., 1922, p. 322.

(*d*).—The requirements for quality of materials, for mixing, transporting, placing, and curing concrete are in harmony with the Joint Committee specification.

(*e*).—Tests are to be carried out to insure that the materials used give the strengths desired.

This form of specification imposes no limit as to the method of proportioning the aggregates. This may be done arbitrarily, by trial, or by calculation based on any of the recognized methods, such as fineness modulus, surface area, maximum density, etc.

The principal advantages of such a specification are that the quality of the concrete is specified on a scientific basis, and that the desired strength is assured, regardless of changes in requirements of workability or in the size and grading of the aggregates. Another advantage is that it makes the interests of the owner and contractor identical. With only the water-ratio specified, the contractor will find it to his advantage to use the largest amount of aggregate consistent with proper workability, thus insuring to the owner the densest and most economical concrete possible for the given strength.

Further, this specification eliminates a source of friction between the contractor and the engineer. If a more plastic mix is desired, it is not obtained at the expense of strength by the addition of water, but by changing the proportion of aggregates to cement or the proportion of coarse to fine aggregates, either of which changes may call for the expenditure of real money on the part of the contractor.

One objection that has been raised to this specification is the difficulty of estimating quantities of materials. This is not a valid objection. After a little experience with this specification the usual tables of quantities can be employed; and in advance of such experience, information of greater accuracy than any tables of quantities may be obtained readily by making a few trial batches.

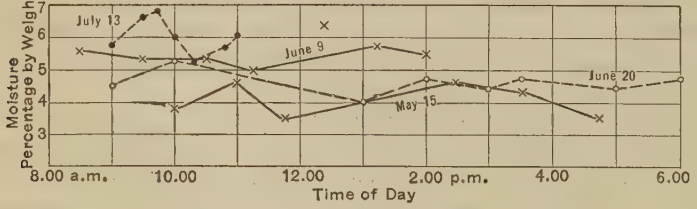


FIG. 2.—VARIATIONS IN MOISTURE OF SAND, FROM TESTS AT CAMDEN, N. J., 1923.

Table 3, discussed under the heading "Data for Estimating" gives information that will be found helpful in making preliminary estimates and that is reasonably accurate for the average materials. Small variations in the quantity of cement are in reality minor items of cost.

It has been suggested that the variation in the moisture in the aggregate might prove a serious objection to this form of specification in view of the fact that the water to be introduced at the mixer must take into account the

moisture in the aggregates. The fears in this respect have likewise proved unfounded. To be sure, there are conditions under which large and rapid fluctuations in moisture content could be expected. Experience has shown, however, that such conditions can be avoided easily; in the normal case the variation will be quite small. In any event, this form of specification is at no greater disadvantage in this respect than any other form yet suggested.

Variations in the moisture content of the aggregate do not occur haphazardly, the moisture being quite uniform throughout any day. Tests on two jobs, each extending over a period of about three months, showed that the moisture in the sand was between 3½ and 7 per cent.* The typical daily variation was about 1% (see Fig. 2). In both these jobs washed sand, shipped directly from the plant, was used.

Experience with Water-Ratio Specification

In actual construction under the water-ratio specification, gratifying results have been obtained. None of the anticipated difficulties has been encountered and practically all the advantages hoped for have been realized. On one job the work has been carried out by two organizations, a general contractor and a sub-contractor, both of whom were thoroughly pleased with this form of specification.

TABLE 2.—Compressive Strengths of Concrete from Construction Job.

Specimens: 6 by 12-in. cylinders.
Samples taken from buggies immediately before concrete was placed in work.
Cylinders cured in damp sand on job until a few days before test; tested damp.
Aggregates: 0 to 4-mesh sand, ¾-in. to 1½-in. crushed stone, measured in wheelbarrows.
Two brands Portland cement used.

Compressive strength desired at 28 days, in pounds per square inch.	Average mix, dry volumes.	Maximum quantity of mixing water specified, U. S. gallon per 94-lb. sack of cement.	Average slump, in inches.	Average Compressive Strength Obtained, in Pounds per Square Inch.		
				3 days.	7 days.	28 days.
2 000	1 : 2½ : 4½	7.5	1½	1 090 (27)*	1 680 (29)*	2 840 (64)*
2 900	1 : 2 : 3¼	6.0	1¼	1 360 (6)*	2 170 (8)*	3 570 (16)*

* Number of specimens tested.

The structural design called for two classes of concrete, one having a compressive strength at 28 days of 2 000 lb. per sq. in. and the other of 2 900 lb. These were specified, respectively, on the basis of 7½ and 6 gal. of mixing water for each sack of cement. In the substructure the coarse aggregate was a crushed limestone graded between about ¾ in. and 1½ in. and in the superstructure a gravel graded between No. 4 and about 1 in., was used. The fine aggregate in both cases was washed bank sand, well graded up to the No. 4 sieve size.

* See Fig. 8, Report on Field Tests of Concrete Used in Construction Work, by W. A. Slater, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., January, 1925, Papers and Discussions, p. 21.

Fig. 3 gives the results of the concrete and aggregate tests made in connection with the substructure. Table 2 summarizes the results of these strength tests and shows the averages of the 3, 7, and 28-day strengths for the two classes of concrete.

These results show that the desired strengths were exceeded by a considerable margin and that a satisfactory uniformity was obtained, considering that no special equipment was used for measuring the materials. All the variations were practically above the required strength. This was undoubtedly due, for the most part, to the great care exercised to insure that the maximum allowable quantity of water was never exceeded. The actual quantity of water was usually $\frac{1}{2}$ to $\frac{3}{4}$ gal. less than that specified.

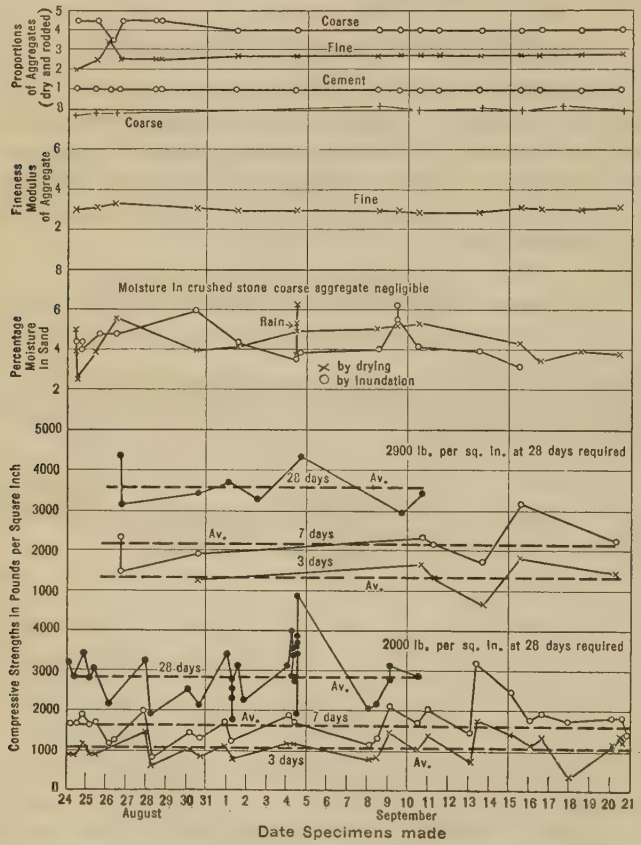

Proportions of Aggregates (dry and rodded)
Coarse
Fine
Cement
Coarse
Fineness Modulus of Aggregate
Fine
Moisture in crushed stone coarse aggregate negligible
Percentage Moisture in Sand
Rain
× by drying
○ by inundation
Compressive Strengths in Pounds per Square Inch
2900 lb. per sq. in. at 28 days required
28 days
Av.
7 days
3 days
2000 lb. per sq. in. at 28 days required
24 25 26 27 28 29 30 31 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21
August
September
Date Specimens made

Fig. 3.—Results of Field Tests of Concrete Made Under the Water-Ratio Specification.

The variation in moisture content of the aggregate during any one day was found to be negligible and the grading of the aggregate was practically the same throughout the job. (See Fig. 3.)

The following observations on the conduct of the job and methods used may be of interest.

The sand and limestone were delivered in truck-load lots and dumped on the street. Storage for more than about a day's work was not available. In spite of this method of handling the materials, practically no important variations in grading occurred.

After the first few batches at the beginning of operations, only a few minor adjustments in proportions were necessary. The moisture content and grading of the aggregates were so uniform that usually the same quantity of water could be placed in the mixer throughout the day, and in several cases, it was not changed for two or three days.

The water was measured in a closed tank directly connected to the city water supply. The tank was equipped with a check-valve which closed when the tank was filled. The quantity of water discharged from the tank was regulated by means of a movable pipe which could be set to give the desired amount. The tank was filled and discharged through a 3-way valve. This type of tank was not found to be particularly satisfactory.

With the incentive of a lower cement factor for the less fluid mixtures, the contractor found it feasible to place much stiffer concrete than that commonly used. This resulted in a definite saving in cement of approximately 1 sack per cu. yd. over that required by the proportions usually specified for that class of concrete.

The average slump was only about 1½ in., but the mass could be readily puddled into place at all times. This avoided segregation and resulted in uniformity throughout each member. No water accumulated on the top, even in the caissons 75 ft. deep, placed continuously.

No inconvenience was caused by the necessity of correcting the quantity of mixing water for the moisture in the aggregate. A simple method was developed for determining the moisture to a fair degree of accuracy within a few minutes without drying the samples, except once for each aggregate to fix the constants.* Check tests by drying and weighing were carried out at the same time. These also were made rapidly and with no great inconvenience.

Data for Estimating

It seems desirable to include data that will give some idea of the range of mixes required for the different water-ratios and be useful in estimating quantities of materials required for concrete of the different proportions. In considering data of this kind, it must be recognized that such generalizations have definite limitations as to accuracy, owing to variations in the grading and bulking of aggregates. The grading cannot be accurately classified by a statement of maximum and minimum sizes. Bulking of the aggregates, vary-

* The following equation gives the basis for the determination of the moisture by inundation:

$$p = 100\,\frac{D - C}{W - D}$$

in which,

p = percentage of moisture by weight;
D = weight of water displaced by moist sample of weight, W;
C = a constant for a given aggregate; it is the weight of water displaced by a dry sample of weight, W; and
W = weight of sample.

ing with the moisture, the method of measurement, and the type of materials, is an important factor. Any set of tables or curves for estimating quantities must be based on certain assumptions covering these variables.

Table 3 is based on the assumption that the gradings are fairly regular between the sizes indicated and that the proportion of coarse to fine material is about as high as is consistent with proper workability. Small changes in this ratio will not make important differences in the quantity of cement required, but are likely to change the consistency noticeably. The proportions are expressed as 1 volume of cement (assumed to weigh 94 lb. per cu. ft.) to a given number of volumes of separated aggregate measured damp and loose. Thus, a 1 : 6 mix in Table 3 represents a field mix of 1 : 2 : 4, 1 : 2½ : 3½, etc.

The curve in Table 3 is taken from Fig. 1 with the water-ratio expressed in gallons per sack of cement. The table is so arranged with reference to this curve, that the mixes which may be expected to produce workable concrete for the different water-ratios are tabulated immediately under their position on the diagram. For example, under the value of 6 in the water-ratio scale, all the mixes listed should produce for the conditions assumed, approximately the slumps indicated when mixed with 6 gal. of water for each sack of cement, and should show a compressive strength at 28 days of 2 900 lb. per sq. in.

Table 3 is not intended for a guide to the actual proportions to be used. These latter should be obtained by selecting the water-ratio corresponding to the strength desired and using fine and coarse aggregate in such combinations as will give the required workability.

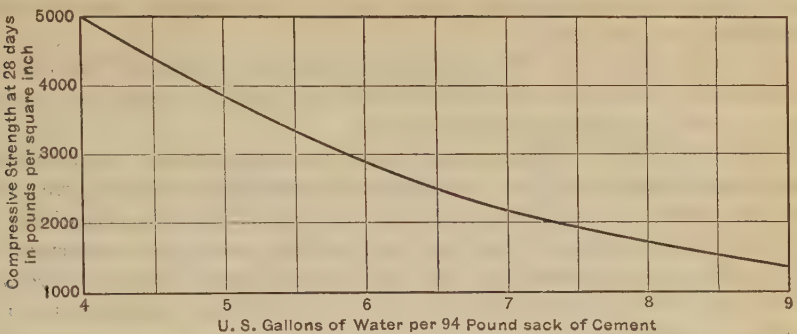
Use of Table 3

The principal purpose of Table 3 is to aid in estimating the quantities of materials required for concrete of a specified strength or water-ratio. The quantities, it will be seen, depend on the size of the aggregate and the consistency, and may vary over a considerable range for concrete of any water-ratio.

Considerable confidence may be placed in the relation between strength and quantity of water shown by the curve, as this has been established by many tests. Also the quantities of cement for the different mixes will be found to be quite accurate for a wide range in type and grading of aggregate. However, considerable variations from the slumps shown may be expected due to minor differences in grading or type of aggregate. These can generally be overcome by adjustments in the proportions of fine to coarse aggregates, or by slight changes in the total volume of the aggregate, without sacrificing quality as long as workable mixtures and proper water-ratios are maintained.

Field Mixes.—The field mixes are given in terms of total volume of separated aggregates measured damp and loose. These values were obtained by assuming the bulking of the sand, due to moisture and to the method of measurement, to be 25%, and of the coarse aggregate, 8 per cent. This method of indicating proportions should not be confused with the method of stating them in terms of volumes of dry and rodded mixed aggregates.

TABLE 3.—Field Mixes and Strengths for Concrete of Given Water-Ratio.



Slump, in inches.	Mix.	Barrels.	Mix.	Barrels.	Mix.	Barrels.	Mix.	Barrels.	Mix.	Barrels.	Mix.	Barrels.
0 to No. 4 Aggregate; Fineness Modulus, 2.75 to 3.25 (100% Sand).												
½–1	1 : 2.0	3.1	1 : 2.8	2.5	1 : 3.5	2.1	1 : 4.1	1.8	1 : 4.7	1.6	1 : 5.2	1.4
2–3	1 : 1.6	3.5	1 : 2.4	2.7	1 : 3.1	2.2	1 : 3.7	1.9	1 : 4.3	1.7	1 : 4.8	1.5
6–8	1 : 1.2	4.2	1 : 2.0	3.1	1 : 2.6	2.5	1 : 3.2	2.1	1 : 3.8	1.8	1 : 4.4	1.6
0 to ⅜-Inch Aggregate; Fineness Modulus, 3.50 to 4.00 (65 to 85% Sand).												
½–1	1 : 2.3	2.8	1 : 3.2	2.3	1 : 4.1	1.9	1 : 4.8	1.7	1 : 5.5	1.4	1 : 6 2	1.3
2–3	1 : 1.8	3.4	1 : 2 7	2.7	1 : 3 6	2.2	1 : 4.3	1.8	1 : 5.1	1.6	1 : 5.8	1.4
6–8	1 : 1.4	3.8	1 : 2.3	2.9	1 : 3.1	2.4	1 : 3.9	2.0	1 : 4.6	1.7	1 : 5.3	1.6
0 to ¾-Inch Aggregate; Fineness Modulus, 4.50 to 5.00 (45 to 65% Sand).												
½–1	1 : 2.6	2.8	1 : 3.9	2.1	1 : 5.0	1.7	1 : 6.0	1.4	1 : 7.0	1.2	1 : 7.8	1.1
2–3	1 : 2.1	3.1	1 : 3.3	2.3	1 : 4.5	1.8	1 : 5.5	1.5	1 : 6.5	1.3	1 : 7.4	1.2
6–8	1 : 1.6	3.6	1 : 2.8	2.7	1 : 3.9	2.1	1 : 4.9	1.7	1 : 5.8	1.5	1 : 6.8	1.3
0 to 1½-Inch Aggregate; Fineness Modulus, 5.25 to 5.75 (30 to 45% Sand).												
½–1	1 : 2.8	2.7	1 : 4.6	1.9	1 : 5.9	1.5	1 : 7.1	1.2	1 : 8.1	1.1	1 : 9.2	0.9
2–3	1 : 2.3	3.1	1 : 4.1	2.1	1 : 5.3	1.7	1 : 6.4	1.4	1 : 7.5	1.2	1 : 8.6	1.0
6–8	1 : 1.8	3.4	1 : 3.2	2.5	1 : 4.4	2.0	1 : 5.6	1.6	1 : 6.6	1.3	1 : 7.7	1.1
0 to 3-Inch Aggregate; Fineness Modulus, 6.00 to 6.50 (27 to 40% Sand).												
½–1	1 : 3.0	2.5	1 : 5.3	1.6	1 : 6.8	1.3	1 : 8.1	1.1	1 : 9.3	0.9	1 : 10.3	0.8
2–3	1 : 2 5	2.8	1 : 4.6	1.8	1 : 5.9	1.4	1 : 7.2	1.2	1 : 8.4	1.0	1 : 9.4	0.9
6–8	1 : 2.0	3.2	1 : 3.5	2.2	1 : 4.9	1.7	1 : 6.0	1.4	1 : 7.2	1.2	1 : 8.3	1.0

Water-Ratio.—The water-ratio is expressed as U. S. gallons per 94-lb. sack of cement. The quantities of mixing water include the moisture in the aggregates; the quantity to be added to the batch is this quantity less the amount carried by the aggregates.

Quantity of Cement.—The quantities under the headings, "Barrels", are in barrels per cubic yard of concrete and apply within the range of proportions of sand to coarse aggregates shown.

Strength.—For the different water-ratios the strengths are based on 28-day compression tests of 6 by 12-in. concrete cylinders puddled in the forms, cured in a damp place at normal temperatures, and tested in a damp condition. In comparing the strengths of concrete made on the work, it is important that the tests be made in accordance with standard methods. Strengths are based on the use of Portland cement meeting the minimum requirements of the standard specifications.

Aggregates.—Table 3 is based on aggregates graded to a fair degree of uniformity between the sizes indicated. The division between the fine aggregate (sand) and coarse aggregate is the square-mesh No. 4 sieve (approximately ¼ in.). The fineness modulus is the sum of the percentages in the sieve analysis, divided by 100, the sieve analysis being expressed as percentages coarser than the following sieves: No. 100, 48, 28, 14, 8, 4, ⅜-in., ¾-in., 1½-in., etc. An important characteristic of these sieves is that the dimension of the square opening of each is double that of the next smaller sieve.

Slump.—The slumps are based on tests with aggregates of rounded sand and gravel of the grading described, using a frustum of a cone 12 in. high, with a top diameter of 4 in. and a bottom diameter of 8 in.

APPENDIX

SPECIFICATION FOR CONCRETE AND CONCRETE MATERIALS AS USED IN THE CONSTRUCTION OF THE PORTLAND CEMENT ASSOCIATION BUILDING, GRAND AVENUE AND DEARBORN STREET, CHICAGO, ILLINOIS.

Suggestions to Bidders

Attention to bidders is particularly called to the following concrete specification which differs materially from the ordinary form in that proportions of aggregates are not specified, while a limiting volume of water per sack of cement is definitely fixed.

This form of specification has been adopted because it has been definitely shown by ample tests and construction experience that it is the proportion of water to cement that determines the strength of the concrete. This means that so long as the ratio of water to cement remains the same, changing the amount of grading of aggregate does not affect the strength, but only the consistency or workability of the concrete; providing only that all mixes are plastic and workable and the aggregates clean and structurally sound.

Under this form of specification the contractor is permitted, within certain limits, to use such aggregates and proportions as in his judgment will pro-

duce proper economy and workability. By keeping the water-cement ratio constant, uniform strength is assured, while the necessary consistency is obtained by varying the aggregate proportions as desired. Increasing the quantity of aggregate for each unit volume of cement, with its fixed ratio of water, increases the yield of the concrete, but stiffens the mass and may increase the cost of placing. Increased plasticity can be obtained by reducing the amount of aggregates used with each sack of cement, or by changing the proportions of fine to coarse.

Bidders may find that in order to obtain the consistencies to which they have been accustomed, more cement will be required per cubic yard than was the case where increased fluidity was obtained by simply adding water. It is, therefore, suggested that trial mixtures be made to determine the cement requirements for the different consistencies which they will use in the various portions of the building. In such trial batches, the moisture contained in the aggregate must be taken into account in securing the water-cement ratio specified.

The following figures indicate how the cement requirement varies with the consistency. For concrete mixed with 6 U. S. gal. of water for each sack of cement, a consistency represented by a 6-in. slump would require about $1\frac{3}{4}$ bbl. of cement per cu. yd. and by a 9-in. slump would require about $2\frac{1}{4}$ bbl. For concrete mixed with $7\frac{1}{2}$ gal. of water for each sack of cement, consistencies represented by a slump of 4 in. would require about $1\frac{3}{8}$ bbl. of cement per cu. yd. and by a 7-in. slump about $1\frac{5}{8}$ bbl. These figures are for average aggregates available in this market. Some variation will be found with different materials or with different gradings.

Attention is further called to the manner in which allowance must be made for batch-to-batch fluctuations in water content. Instead of specifying the average water-cement ratios desired with a definite tolerance, the maximum permissible values have been given, requiring the contractor to so conduct operations that those shall at no time be exceeded. This will enable the contractor to obtain increased economy as his methods of controlling the water are perfected, for the smaller the variation, the closer the maximum permissible limits can be approximated by the average water content.

The principal difficulties of controlling the mixtures under this type of specification will arise from variations in moisture content and grading of the aggregate, and from inaccuracies in measurement—principally with the sand. The requirement in the specifications, that the source of aggregate be such that uniformity of grading can be assured during any one day's operations, will largely eliminate some of the difficulty. This requirement will be found decidedly to the contractor's interest and one that in this market should offer no hardship. If methods of measuring aggregates are adopted which will give uniform quantities from batch to batch, not only will other difficulties be eliminated, but the annoyance of moisture variation in the sand will be considerably minimized. For with uniformity in quantities of materials and grading, the correct moisture content in the concrete can be gauged with considerable accuracy by the workability or consistency as indicated by the slump test or by its appearance.

Determinations of the moisture content in the aggregate from time to time will be necessary in order that the water content will at all times be within the water-cement ratios specified. The frequency with which these determinations will be required and the consequent annoyance of making the requisite changes in the mix will depend entirely upon the care used in controlling the grading and uniformity of measurement and the moisture content in the aggregates.

The measurement of moisture in the aggregate within the limit of accuracy specified (2 lb. water in 100 lb. aggregate) should offer no great difficulty.

Either drying and weighing a small sample or by inundation will give results satisfactory for the purpose.

Since this is likely to be a new form of specification to the bidder the owners will have on the job or immediately available at all times a man experienced in work of this character who will be ready to advise and assist the contractor in developing methods of carrying out the specifications and in overcoming any difficulties that may arise.

Specifications for Concrete and Concrete Materials

Water-Cement Ratio.—Concrete for structural members shall be proportioned to give the necessary workability without exceeding the following ratios of water to cement:

Where concrete of strength of 2 900 lb. per sq. in. is called for:
6.0 U. S. gal. water per sack (94 lb.) of cement.
Where concrete of strength of 2 000 lb. per sq. in. is called for:
7.5 U. S. gal. water per sack (94 lb.) of cement.

These water-cement ratios are the maximum permissible. The mixes shall be proportioned for somewhat lower ratios so that with the normal fluctuations which may be expected from batch to batch these ratios will not be exceeded. Water or moisture contained in the aggregate must be included in computing the water-cement ratios. Water absorbed by the aggregate in a period of 30 min. may be deducted in computing the water-cement ratio.

The water-cement ratios specified shall not be changed except by the Architect. In the event that the Architect finds it necessary to change the water-cement ratios from those specified, adjustment, covering amount of cement and aggregates affected, will be made as an extra or a credit under the provisions of the contract.

Measuring Moisture in the Aggregate.—Moisture in the aggregate shall be measured by a method, satisfactory to the Architect, which will give results within 2 lb. for each 100 lb. of aggregate.

Concrete Proportions and Consistency.—The proportions of aggregate to cement for concrete of the water-cement ratios specified shall be such as to produce concrete that can be puddled readily into the corners and angles of the form and around the reinforcement without excessive spading and without undue accumulation of water or laitance on the surface. In no case shall concrete be placed which shows a slump exceeding the following limits:

For caissons.....Maximum slump, 4 in.
For heavy walls, slabs, and beams..... " " , 7 in.
For thin walls and columns..... " " , 9 in.

The proportion of fine and coarse aggregates shall be such that the ratio of the coarse to the fine shall not be less than 1 nor more than 2, nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the surface and corners of the members shall be smooth and sound throughout.

Control of Proportions.—The methods of measuring materials shall be such that the proportion of water to cement can be closely controlled during the progress of the work and easily checked at any time by the Architect or his representative.

To avoid unnecessary or haphazard changes in consistency, the aggregates shall be obtained from a source which will insure uniform quality and grading during any single day's operation, and they shall be delivered to the work and handled in such a manner that variations in moisture content will not interfere with the steady production of concrete of a reasonable degree of uniformity.

Tests of Concrete.—Frequent tests will be made by the Architect throughout the work to determine the quality of concrete being produced. These tests

will be made at the expense of the owner and will, in general, be made on 6 by 12-in. concrete cylinders loaded in compression at ages of 7 and 28 days, in accordance with the Standard Method of Making and Storing Specimens of Concrete in the Field (Serial Designation: C31-21) of the American Society for Testing Materials.

The contractor shall co-operate in every way to the end that concrete of the desired quality be obtained. He shall provide at cost, such housing as may be required for testing equipment and storage of test specimens; such cost to include only labor and materials actually used.

Portland Cement.—Portland cement shall conform to the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials (Serial Designation: C9-21).

Fine Aggregate.—Fine aggregate shall consist of natural sand having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam, or other deleterious substances. The sand shall be of such sizes that it shall all pass a ⅜-in. sieve, at least 15% shall be retained on the No. 8 sieve, and the fineness modulus shall not exceed 3.50.

Coarse Aggregate.—Coarse aggregate shall consist of gravel, crushed stone, crushed air-cooled blast-furnace slag weighing not less than 70 lb. per cu. ft. or other approved inert materials, having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated, or laminated pieces, alkali, organic or other deleterious matter. Coarse aggregate shall not have more than 10% finer than the No. 4 sieve and the maximum size shall not be greater than will permit proper placement.

Storage of Aggregate.—Aggregate shall be so stored as to avoid inclusion of foreign materials. Frozen aggregate or aggregate containing lumps of frozen material shall be thawed before using.

Water.—Water for concrete shall be from the Chicago water supply or other approved source.

Mixing Concrete.—The concrete shall be thoroughly mixed in a batch mixer of approved type. The mixer shall be equipped with suitable charging hopper. A water-storage and water-measuring device shall be provided. The mixing of each batch shall continue for at least 1 min. after all the materials are in the mixer during which time the mixer shall rotate at a peripheral speed of approximately 200 ft. per min.

Depositing Concrete.—Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable and in a manner that will prevent segregation of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. Concrete as it is deposited shall be puddled by means of suitable tools until forms are completely filled and reinforcement and embedded fixtures thoroughly incorporated in the mass.

Water shall be removed from excavations before the concrete is deposited, unless otherwise directed by the Architect.

Concrete when deposited shall have a temperature of not less than 40° Fahr. and not more than 120° Fahr. Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation, approved by the owner's representative, is completed.

In freezing weather suitable means shall be provided for maintaining the concrete at a temperature of at least 50° Fahr. for not less than 72 hours after placing, or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the Architect. Salt, chemicals, or other foreign materials shall not be mixed with the concrete for the purpose of preventing freezing.

Protection of Concrete.—Exposed surfaces of concrete shall be protected from drying for a period of at least 7 days after being deposited.

DISCUSSION

C. R. F. Coutlee,* M. Am. Soc. C. E.—In the fall of 1886 the speaker first saw concrete placed. It was in the bottom of a small bridge abutment in Beaudette River. First, 12 by 12-in. timbers were laid 1 ft. apart and then some natural cement concrete was placed between them. On top of these, other timbers were laid crosswise, and the spaces similarly filled; the quantity of water was not excessive. The timber was considered necessary to give strength to the concrete.

In the Nineties more than 200 000 cu. yd. of concrete were placed in the Soulanges Canal works. Absolutely the greatest quantity of water that could be put into it was used, and it is still standing, in fact, it has been referred to as among the best examples of concrete. Of course, a porous sandstone was used, which absorbed water freely and that must have corrected the water-cement ratio. Only mass concrete was used, the weight being taken at 125 lb. per cu. ft.

The sand was measured without regard to moisture and 10% more than the voids of the stone was used. It was clean, of fairly large and strong grain. European cements, mostly Belgian, were used in Canada then; at present, there is one big cement company in the country, and the product is, like Caesar's wife, beyond suspicion.

In placing concrete under water for walls on top of cribwork, the general practice was to let it set in the skips for an hour or more. The area was divided by false forms, and the skip emptied slowly to form an end dump. Canvas aprons were tacked to the forms if there was wave action.

A. S. Woodle, Jr.,† M. Am. Soc. C. E. (by letter).—The chief difficulty in obtaining general recognition and adoption by engineers of the scientific method of manufacturing concrete as proposed by Professor Abrams in his water-ratio theory, has been that of educating the profession in the theory sufficiently to convince its members of the validity of the tests and the soundness of the conclusions made from them. This recognition has been gradually brought about by the frequent discussion in the technical press, at meetings of technical societies, and by aggressive campaigning on the part of the Portland Cement Association, until it has become the exception rather than the rule to see a concrete job of any magnitude built by the old-fashioned slipshod "rule-of-thumb" methods.

There has, however, been one deterrent condition in effect, which has retarded this desirable movement, that is, the lack of a standard specification by which ordinary concrete mixes in every-day use may be proportioned without the assistance of a concrete testing laboratory. In the writer's opinion, the specification submitted by the authors is a splendid first approximation to such a standard specification; in fact, if it were extended to include both weaker and stronger mixes than those shown, there would be little left to criticize.

* Ottawa, Ont., Canada.

† Engr. of Plant, Baldwin Locomotive Works, Philadelphia, Pa.

A table giving the slump to be expected from various combinations of water-cement ratio to coarse and fine aggregate ratio would be very useful. Also, the specification would be more complete if mention were made of admixtures, particularly calcium chloride, which seems to be coming into favor as an agent for curing the concrete, and for improving the strength at all ages.

One stumbling block which crops out several times in this paper, and against which every student of concrete proportioning has stubbed his toe, is the need for a clear-cut scientific definition of, and test for, the very elusive property, "workability". He who successfully evaluates this will-o'-the-wisp variable—which is a function of the water ratio, of the cement aggregate ratio, of the fine aggregate-coarse aggregate ratio, of the fineness modulus of the aggregate (both coarse and fine), probably also of the shape and texture of the aggregate, and perhaps of many others—will have taken the next great step forward in the scientific manufacture of concrete.

T. P. Watson,* M. Am. Soc. C. E. (by letter).—It is the writer's opinion, in agreement with some authorities, that the quality of concrete is dependent principally on the quality of the mortar, or the combination of water, cement, and fine aggregates. He does not agree with the authors in their hypothesis that:

"* * * the fundamental principle that the strength and other desirable properties of concrete are definitely determined by the proportion of the water to the cement in the mixture, provided only that the concrete is plastic and workable and the aggregates are clean, durable, and structurally sound."

This theory is not substantiated by the data presented. A comparison of the results in Table 1 with those represented in Fig. 1 discloses differences that are typical of the inherent futility of a concrete strength specification for general application because of the different strengths obtained by the use of different cements and different types of aggregates, and on account of the personal equation in the making and curing of test specimens. The data contained in Table 4 were taken from Table 1.

TABLE 4.—Compressive Strengths of Concrete for Various Water Ratios.

Mixture by volume, in terms of dry rodded volumes of aggregates.	Quantities of Mixing Water (U. S. Gallons) per Sack (94 Lb.) of Cement.				
	6.	6½.	7.	7½.	8.
1:1.5:0	4 300	3 970	.....	.....	....
1:2 :0	4 090	3 740	3 340	.....	....
1:2.5:0	3 560	3 510	3 430	3 210	....
1:3 :0	.....	.....	.....	2 980	....

It will be noted from Table 4 that there was a general decrease in strengths as the proportions of fine aggregates were increased, even if the same water-cement ratios were maintained, and that when a proportion of 1:2.5 was

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reached, the water-cement ratios affected the strengths in a much lessened degree.

Further analysis of Table 1 discloses that there was a definite relation between the maximum concrete strengths for the various water-cement ratios and particular proportions of fine and coarse aggregates. Also, for a given water-ratio and fixed proportions of cement and fine aggregate, the introduction of a small quantity of coarse aggregate almost invariably decreased the strengths; and increased proportions of coarse aggregates produced higher strengths until proportions were reached at which the maximum strengths were obtained. Thereafter further increase of the proportion of coarse aggregate resulted in lower strengths.

The lower strengths obtained from mixtures containing the smaller proportions of coarse aggregates were caused by the introduction into the mortar of planes of cleavage along the surfaces of the particles of the coarse aggregates. The higher strengths obtained from mixtures containing larger proportions of coarse aggregates were caused by the absorption into the coarse aggregates of a part of the water and by the phenomena of the curing of the concrete. This was aided by the available moisture temporarily stored in the coarse aggregates, thereby increasing the mortar strengths sufficiently to overcome the structural weakness of the concrete caused by the additional planes of cleavage due to the increased proportions of coarse aggregates. When the proportions of coarse aggregates were increased beyond the point where the mortar filled the voids the strengths became less.

Compressive strength has answered a useful purpose in spurring concrete users to strive for the better properties of concrete; but compressive strength is far from the basic essential quality required for the vast majority of concrete structures. The quality most desired is sufficient durability to withstand the destructive agents to which the concrete will be exposed.

Lasting impermeability is probably the cardinal quality to be sought for durable concrete. Research regarding permeability usually proves that the least permeable concretes were made with comparatively fine sands, yet the paradoxical tendency of concrete authorities is to advocate the use of the coarser gradings of sands because of the higher strengths obtained.

The same reasoning may be used with gradations of fine aggregates as was used previously in referring to concrete strengths. The higher strengths obtained with the coarser gradings of fine aggregate are due to the absorption and phenomena of curing, and to the relatively rich matrix of those fine particles of the fine aggregate in combination with the water and cement, in which are suspended the larger particles erroneously assumed to be fine aggregate, but which, in reality, when larger than some unknown size, are found to be parts of coarse aggregate.

The question of the maximum permissible allowance of fine aggregate passing a No. 100 sieve is pertinent when considering durability and is a matter too frequently overlooked. Fine aggregates with any considerable percentage passing a No. 100 sieve should be particularly investigated. If no other aggregate is available a compensating increase in the proportion of cement should be made, because in the usual run of fine aggregates the pro-

portion that passes a No. 100 sieve is practically an inert powder which acts as an adulterant to the cement and produces a less durable mortar.

It is generally conceded that concretes of very low strengths and other theoretically undesirable properties have given satisfactory service when not exposed to outside climatic conditions, frequent alternate wetting and drying, hydrostatic pressures, or other destructive agents. It would seem logical, therefore, that concrete for particular purposes should be specified in accordance with the conditions of exposure and durability.

The writer is in agreement with the authors that the water-cement ratio is a basis for concrete specifications. The principal purpose of the suggested amendments to the proposed specification is as follows: (*a*) The avoidance of as many technical terms as possible; (*b*) the avoidance of a strength specification; (*c*) the fixing of definite proportions of water, cement, and fine aggregate; and (*d*) the sanction of the proper use of smaller gradings of fine aggregates. The following amendments to the proposed specification are suggested:

Omit in its entirety the Section entitled "Suggestions to Bidders".

Water-Cement Ratio.—Omit this Article and substitute the following:

"*Concrete.*—Concrete shall be placed in accordance with classifications indicated on the plans and the following proportions of water, cement, and fine aggregate (dependent on the size classification hereinafter described) shall be combined with such proportions of coarse aggregate as will produce a mixture that can be readily puddled in the forms and around reinforcement without segregation and that will insure surfaces free from honeycomb when forms are removed:

Concrete classification.	Proportions.					
	Water, in U. S. gallons.	Cement, in sacks (94 lb.).	* Fine aggregate, in cubic feet saturated.			
			Size classifications.			
			No. 4.	No. 8.	No. 16.	No. 30.
A	†5.0	1.0	1.3	1.3	1.0	0.9
B	†6.0	1.0	1.6	1.6	1.3	1.2
C	†7.0	1.0	2.0	2.0	1.7	1.5
D	†8.0	1.0	2.4	2.4	2.1	1.9

* If fine aggregate is measured in a batcher or other receptable by loose volume its loose volumetric relation to a saturated volume shall be determined.

† Moisture contained in the aggregates must be deducted.

"The method of measuring materials shall be such that all proportions are closely controlled and easily checked at any time.

"Aggregates shall be such as will insure uniform quality and gradings.*

"*Concrete Classifications.*—

"(*A*) Thin reinforced slab sections exposed to outside climatic conditions.

* With a definite quality of mortar the coarse aggregate acts as an economical filler and as long as the proportions are such that the mass is readily placed without segregation there will always be sufficient mortar to fill the voids.

"(*B*) Reinforced beams or columns, thin reinforced walls, copings, projections, water-tables, or horizontal surfaces on which water is likely to accumulate when exposed to outside climatic conditions or especial interior conditions.

"(*C*) Interior structural members; and structures or structural members not reinforced, with comparatively vertical surfaces, when exposed to outside climatic conditions.

"(*D*) Foundations not exposed to outside climatic conditions."

Measuring Moisture in the Aggregate.—This Article should be amended to restrict variance in moisture so as not to exceed 1 lb. in each 100 lb. of aggregate. The allowance of 2 lb. is excessive and would defeat the application of the water-cement theory.

A clause should be inserted to prevent the allowance of any additional water on account of the absorption by the coarse aggregate. Coarse aggregate of such porosity as will absorb an appreciable quantity of water should be surrounded by a mortar of a lower water-cement ratio to offset the permeability of this type of aggregate. If no allowance for absorption is made in the quantity of water the capillarity of this type of aggregate to some extent automatically accomplishes the desired result.

Concrete Proportions and Consistency.—This Section should be omitted. The personal equation entering into the making of slump tests is such that, in especially qualified hands, the slumps are only a guide to the water content, when exacting methods of proportioning are used to insure identical quantities of all proportions of uniformly graded fine and coarse aggregates. The slump test should be avoided as part of a specification.

Control of Proportions.—This Article should be omitted.

Tests of Concrete.—This Section should be omitted. Although it is desirable that frequent tests should be made to satisfy the Architect or Engineer of the uniformity of the concrete as accurately gauged by concrete specimens when properly made and cured, they are not pertinent to the specifications.

Fine Aggregate.—This paragraph should be amended by omitting the second sentence and substituting the following:

"Fine aggregate shall be within one of the following size classifications:

Size Classification.	To Pass.	Retained at Least.
No. 4	3/8-in. sieve 100%	No. 8 sieve 15%. No. 100 sieve 97%
No. 8	No. 4 sieve 100%	No. 16 sieve 15%. No. 100 sieve 97%
No. 16	No. 8 sieve 100%	No. 30 sieve 15%. No. 100 sieve 97%
No. 30	No. 16 sieve 100%	No. 50 sieve 15%. No. 100 sieve 97%"

Mixing Concrete.—This Section should be amended to increase the minimum mixing interval to 1½ min.

Depositing Concrete.—The third paragraph of this Article should be amended to limit temperatures of not less than 50° Fahr., or more than 90° Fahr.

Experience has shown that under certain seasonal conditions the placing of concrete at a temperature as low as 40° Fahr. involves unnecessary risk. The maximum temperature of the concrete should not be above 90° because

when the concrete temperature is higher there is danger of structural complications from too hasty setting. The maintenance of a temperature of about 70° Fahr. will give very satisfactory results.

CHARLES S. WHITNEY,* M. AM. SOC. C. E. (by letter).—The writer is not yet convinced of the necessity or advisability of the use of a specification for concrete that does not give the contractor definite proportions on which to base his estimate of cost. While the method of technical control of proportioning may be explained in the specification so that the contractor will know what to expect, the administration of it should be entirely in the hands of the engineer. The purpose of any specification should be to insure to the owner a good job and to the contractor a reasonable compensation. If the control of the proportioning is under the direction of the engineer employed by the owner, the saving effected by such control should go to the owner. In other words, the owner should pay a reasonable price for the materials and labor, and the contractor should be relieved in so far as possible of the gamble which results from an indefinite specification. If competition is desired, it is fair that the contractors should estimate on the same quantities, so that the contract will not go to the one who happens to be the most optimistic about the strength of his concrete.

For uniform bidding, the specification should give definite instructions as to the quality of materials and the proportions to be used. It may then be provided that the engineer may establish the exact proportions by any desirable method after samples of the materials have been submitted and during construction, and that the owner is to be credited or charged with the difference in cost of the materials. It is also well to specify as accurately as possible the consistency desired, that there may be no misunderstanding. Such a specification leaves the engineer entirely free to proportion the concrete in any way he may think best without depriving the contractor of any of his profit and makes possible the most efficient use of the materials and of the owner's funds.

If the strength of concrete depended entirely on the water-cement ratio, the selection of aggregate would be very easy, but the problem is not as simple as that. The only disadvantage of the form of specification suggested by the writer is that the owner will not know in advance exactly what the total cost will be. It should be possible, however, for the engineer to predict the proper proportions closely enough, so that the change made during construction will have little effect on the cost. The owner will pay only for what he receives, and the contractor will not have a financial interest in using the cheapest possible mix. In case the owner is not represented by an independent engineer and the proportioning is in the hands of the contractor's force, it would seem desirable to include in the specification some form of strength clause. Even then the payment could be based on the actual proportions used.

HARRY C. BOYDEN,† M. AM. SOC. C. E.—The tests mentioned in the paper invariably show higher results than the strengths designed for. Is it not true

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that the curve, $S = \frac{14\,000}{9^x}$, was used in designing the mixture rather than $S = \frac{14\,000}{7^x}$?

There has been considerable confusion in this matter. The original curve published by Professor Abrams was $S = \frac{14\,000}{7^x}$, which indicated the average strengths obtained with varying values for $\frac{w}{c}$ (water-cement ratio). In later publications the curve, $S = \frac{14\,000}{9^x}$, has been used, which gives the minimum strengths obtained with varying values for $\frac{w}{c}$. When this curve is used in designing mixtures the actual results nearly always average higher than are called for by the design and this has caused confusion and comment.

A word in defense of the slump test may not be out of place as the authors have rather discouraged its use. Although this is not a perfect test of strength, nevertheless it has its place in controlling the quality of concrete. It has been demonstrated many times that it will serve as a check against the use of varying quantities of water in successive batches. It is possible on almost any job of concreting to improve the quality by using the slump test. The procedure is to establish the consistency desired or needed for that particular job, to find the slump, and then to keep all batches at the same slump. This will give a much better control of the quantity of water used than is found on the average job.

There is a record of a famous dam where the quality of the concrete was more than doubled by a much cruder method than the slump test. During the first year of construction the only control of consistency was by keeping the concrete stiff enough to prevent a man from sinking into it over the tops of his rubber boots. Then the rule was changed and orders were issued that he must not sink in more than 10 in. Immediately the compressive strength of the concrete, as shown by daily cylinders, increased more than 100 per cent. This might be called an "inverted slump test," but it was certainly effective.

L. A. Perry,* Assoc. M. Am. Soc. C. E. (by letter).—This water-ratio specification for concrete has been used by the writer in the construction of approximately 95 000 sq. yd. of concrete pavement during 1926. This method of control was first considered when the writer discussed the subject with Mr. McMillan, in February, 1926. At that time the writer raised many objections to it. However, a fair trial has revealed the pronounced advantages of this control and has established the paper as an extremely important presentation.

Those who realize the importance of the limitation of mixing water and who know that the tendency toward an excessive quantity of water is the

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prevailing sin in pavement construction, must readily appreciate a specification which induces the contractor through personal financial interest to avoid such excess. Water-ratio control does this in a very practical way and also gives reasonable assurance of a predetermined strength of concrete. Under this specification it also behooves the contractor to select materials of good grading and to avoid stock-pile segregation. The necessity or desirability of inundation of sand ceases to exist. The slump-tube and flow-table are unnecessary. There is no occasion to fix any given sieve analysis or fineness modulus or to specify any given quantities of aggregates. All that is necessary is to specify the water and cement contents of the mixture (or rather the ratio of water to cement) and that the aggregates be clean and of sound grain.

The authors' Table 3 gives a wide range of mixtures. The 5 gal. per sack mix has been used by the writer, and test cylinders from this mix checked closely with the strength predetermined by the chart in Table 3. An average of 3 625 lb. per sq. in. was developed by cylinders tested with lumnite caps; and 3 452 lb. per sq. in. was developed by cylinders tested with plaster-of-Paris caps. Paper moulds were used, cylinders of which do not ordinarily break as high as those cast in machined steel moulds.

The specifications for the work are quoted in part as follows:

"Portland cement concrete pavement shall be composed of a mixture proportioned by parts, as follows: 5 (five) U. S. gallons water, 1 (one) sack cement, and as much fine and coarse aggregate as the Contractor may desire to use; providing, however, that the mixture is at all times, in the opinion of the Engineer, of desirable workability.

"The materials shall be as specified in the 'Standard Plans and Specifications of the City of Longview, Revision of April 1st, 1925'; excepting that the greatest dimension of aggregates shall not exceed ½ (one-half) the depth of the pavement, and that the grading of aggregates within the above limitation shall be optional with the Contractor.

"The amount of mixing water herein specified (5 U. S. gallons per sack of cement) shall include the moisture actually contained in the aggregates used; and the amounts of water charged into the batch shall be adjusted in such manner that the moisture in the aggregates and the water charged into the batch shall not, together, be more than the specified water content of mixture (5 U. S. gallons per sack of cement).

"Such adjustments of the water charged into the batch shall be made in accordance with tests conducted by the Engineer, or by his assistants, as frequently as he may deem necessary to determine the moisture content of the aggregates used.

"In the making of such tests and adjustments, the following assumptions shall be made:

"Aggregates weigh 110 lb. (one hundred and ten pounds) per cubic foot.

"The amount of fine aggregate used per sack of cement will be two (2) cubic feet.

"The amount of coarse aggregate used per sack of cement will be four (4) cubic feet.

"It is expressly specified that even though the Contractor may elect to vary the amounts of fine and/or coarse aggregate per batch, the above stipulated amounts of aggregates shall form the basis for adjustment of mixing water charged into the batch (or batches)."

The method of moisture determination was designed for simplicity and definiteness to preclude argument with contractors. All calculations were avoided; a 500-gramme sample of sand was selected by weight and dried, then weighed again on a metric scale. The loss of moisture was read directly on the scale. A double electric hot-plate provided at the bunker and a number of pans enabled the inspector to make tests almost continuously although changes were remarkably infrequent. The time required to make each test was about 5 min. The moisture content of coarse aggregates was determined in the same manner except that it was found desirable to select a sample of somewhat finer grading than was representative in order to offset the tendency of large pieces to dry and lose moisture before the first weighing.

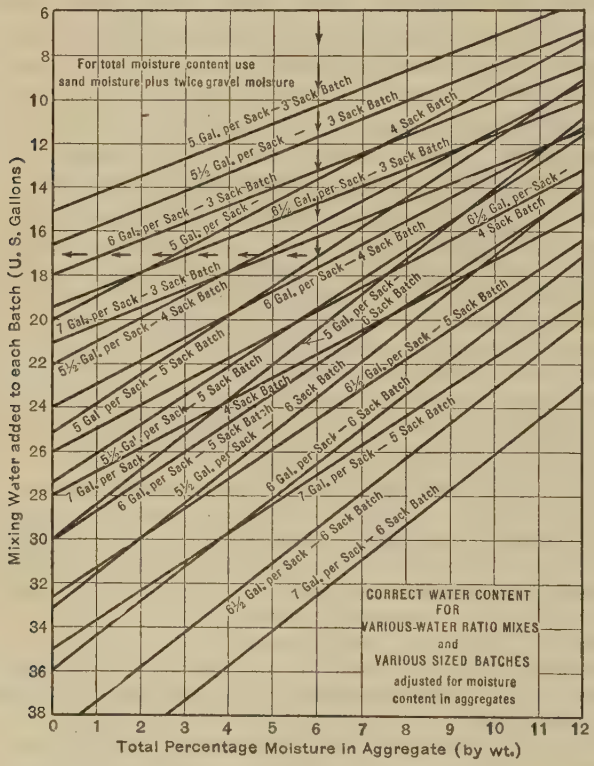

FIG. 4.

A chart (Fig. 4) was used to determine the quantities of mixing water to be added for various moisture contents of aggregates. For simplicity this chart is based on the total moisture so that it becomes necessary to combine the fine and coarse aggregate moisture, to arrive at the total, when, as in the case of the foregoing specifications, 1 : 2 : 4 proportions are assumed for a basis of adjustments. The total moisture is then the sand percentage plus twice the gravel percentage; for instance, if 4% water is found in the sand and 1%

in the gravel, the total percentage used would be 6. Following the chart it is found that the correct quantity of mixing water to be added in this case for a 5-sack batch of 5 gal. per sack concrete would be 17.1 gal. and that the remainder of the 25 gal. (7.9 gal.) is retained in the aggregates. Under some conditions it was believed desirable to sprinkle the gravel stock-pile to keep the moisture content uniform. On isolated work, where this control cannot be kept at bunkers, drying can be conveniently done over an open wood fire, at the immediate site of mixing and deposit.

In the writer's practice it has not been found necessary to make any reservations regarding sieve analysis or fineness modulus of either fine or coarse aggregates or to set any limiting ratio of fine to coarse, as suggested by the authors. Any marked departure from density gradation was keenly felt by the contractor and was remedied very quickly. The bulking effect of sands renders any fine-to-coarse ratio limitation unreliable. It is a comforting fact that the water-ratio theory automatically takes care of sand bulking, and gradation.

It is thought that in the authors' specification a typographical error exists where it is stated that "moisture in the aggregate shall be measured by a method * * * which will give results to within 2 lb. for each 100 lb. of aggregate", as this would permit a variation of about 1½ gal. per sack of cement in a 5 gal. per sack mix. This degree of accuracy of moisture measurement is not satisfactory, at least for the best grades of concrete. By the method described in this discussion, moisture measurements were made to within 0.25% by weight (¼ lb. to each 100 lb. aggregate), which permits a variation of ½ gal. of water per sack of cement.

The writer has heard many objections to water-cement-ratio control, chief among which has been the alleged possible abuse of a seemingly lenient specification, by the use of an excessively lean, dry mix that does not get proper placement. It need only be remembered that economical management of construction does not permit such a thing. In order to save fifty cents in cement by such abuses it would be necessary to spend an extra dollar in labor cost. The writer has seen this demonstrated. It is perhaps a measure of safety, however, to include the reservation in the first paragraph of the specifications quoted in this discussion, thus, "providing however that the mixture is, at all times, in the opinion of the Engineer, of desirable workability."

A full appreciation of the value of this method of mix control will be realized when it is remembered that it eliminates the use of inundation, sieve analyses, fineness moduli, slump-tubes and other consistency gauges, aggregate measurement, and much argument; it is a more simple reliable control than specifying actual or nominal mixes; and it is more equitable to the contractor who is willing to do some of his own inspecting.

This discussion concerns the application of water-ratio specifications to pavement construction where the need of some such control has long been felt.

JOHN G. AHLERS,* M. AM. SOC. C. E. (by letter).—The general criticism of this paper would be that the specification is too strict in not allowing enough latitude for the contractor who will honestly endeavor to make concrete of a given strength so that he may benefit from the resulting economies. Particularly is it a handicap where the water ratios are restricted to the maximum permissible, if the strengths are produced uniformly and within limited tolerances. A contractor should be permitted to change the water-cement ratio and obtain the benefit from his supervision and control methods.

The specification is correct in allowing the proportions of aggregate to cement to be governed only by the requirements of workability, but it should go further and allow a change in the water-cement ratio where proper careful control methods are used to keep this strength.

The specification brings out the point that has been stressed very much in the writer's own practical contracting experience that the object of owner and contractor can be made identical and the use of the water-ratio specification is the greatest step forward in harmony between the Contracting and Engineering Professions in practical field work. This has been particularly stressed by the writer,† based on actual experiences on twenty building operations where water-ratio control was used on all concrete placed in the structures.

It has also been the writer's experience that there is no difficulty in estimating the quantities of material used under control by the water-cement ratio. After one or two operations every contracting organization will arrive at its own constants of cement factors for different strengths and for different types of materials used.

Collection of these data alone is of great economic value to the contractor and ultimately to the engineer, architect, and owner, because when these economies are realized the contracting field will be able to produce better concrete at lower prices than in the past.

For water-ratio control the ordinary type of tank is not satisfactory. The special equipment used by the writer has previously been described.† No inconvenience has been caused by the necessity of correcting the quantity of mixing water for the moisture in the aggregate.

Perhaps the chief value of water-ratio control lies in that both contractors and engineers are made to study the available cement and aggregate more thoroughly and thus to get a different perspective on the factors entering into the production of good concrete. The following improvements in the specification are suggested:

(*a*) Instead of specifying the maximum permissible water-cement ratio, exact water-cement ratios should be specified, based on actual tests of concrete produced under job conditions.

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* Secy.-Treas., Barney-Ahlers Constr. Corporation, New York, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 823.

(*b*) Portland cement should be tested for compression as well as for tension as this would give a definite indication of its desirability and of the strength that may be expected for certain water-cement ratios. A specification for tests of Portland cement would be a good indicator of the water-cement ratios to be specified for the work. Then the time of mixing need not be limited; this factor is already covered.

THADDEUS MERRIMAN,* M. AM. SOC. C. E.—The authors are to be commended for their frank recognition of the fact that compressive strength bears no relation to either the permanence or the durability of concrete. They have presented one of the best practical statements on the design of concrete mixes that has yet been prepared.

Quite aside from "the design of concrete mixtures" there is another aspect of the problem of concrete, which demands attention. The wheels of progress seem to have revolved to that point on the road to progress where reasonable agreement in regard to the matters of water-ratio and proportions is indicated; but this point does not mark the journey's end. Because agreement has been reached on the obvious things, which are visible to the eye, it does not follow that the problem has been completely solved. From these gross physical manifestations the inquiry must go forward to a consideration of the conditions under which the cement reactions occur and seek to ascertain how it will be possible to secure both an optimum of concrete quality and a maximum of permanence and durability.

A practical demonstration will serve to illustrate differences in quality of product. Mix a sample of cement with 43% of water by weight. Pour one-half this mixture into one bottle, the other half into another. Break the first bottle at the end of 24 hours and allow the sample to remain in the air. The second bottle should be broken when 21 days old and the sample thereafter kept in the air. These two samples will speak for themselves. The first will be soft, porous, and chalky, while the second will be hard, dense, and of rock-like structure. Made from the same cement, they will look like two entirely different products. The chemical reactions in the one sample will evidently have been of a very different order from those in the other.

The important question to be determined is that of learning why concrete in one place is good and strong while in another it disintegrates and shows little of either permanence or durability. The water-cement ratio seems to have some bearing on this matter, yet the day is not far distant when concrete will be designed on the basis of the proportionate relationships between (*a*) the cement and the water; and (*b*) the cement and the sand. Then, further, consideration must also be given to the environment in which the cement undergoes its hydrations. These conditions being under control, the user may then put in as much or as little stone as he desires.

* Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

Permanent concrete can hardly be designed on the basis of the water-cement ratio only. Of the many factors that go to determine the quality of concrete this ratio is only one, and it would be passing strange if this simple and obvious arithmetical fraction should constitute the last word in the act of concrete making. The factor of cement quality is not so easily to be submerged. Until the solution concentrations in which the cement reactions occur are held under control the nature of the hydration products cannot be predicted. These products are those which impart to concrete its well-known but little understood characteristics. Until knowledge of cement and its behavior is vastly greater than it now is, but little more can be developed than is now known in regard to the mixture called concrete.

F. R. McMillan,* M. Am. Soc. C. E., and Stanton Walker,† Assoc. M. Am. Soc. C. E. (by letter).—The discussion of "the water-ratio specification" has brought out a number of features which add to the interest of the subject and contribute information of permanent value. In practically all the discussion the evident intent of the contributor is the same as that of the writers. The writers believe that some of the points were covered in the original discussion, while others require some further amplification to avoid confusion. The amount of interest shown in the paper is most gratifying.

Mr. Coutlee emphasizes the important point that the absorption of the aggregate is of considerable importance in determining the net quantity of mixing water. It frequently occurs also that concrete is improved in quality by the leakage of water from the forms, loss through evaporation, etc., during placing or shortly thereafter. It must be remembered that it is the water-cement ratio in the mass as it is solidified in the forms that actually determines the potential strength of the concrete. In the specification accompanying the paper it was recognized that the quantity of water absorbed in a period of 30 min. may be deducted.

Mr. Woodle points out that a table of proportions of materials for different water-ratios and gradings of aggregate would be desirable. The writers recognize the convenience of such a table, but feel that it would defeat, to a great extent, one of the aims of the specification—that of flexibility in the selection of materials and proportions. He also suggests that the specification would have been more complete if it had covered the use of admixtures. This specification was not intended to cover all the special applications of concrete, but rather to cover the basic factors pertaining to proportioning to meet definite strength requirements in the usual case.

The writers recognize the point made by Mr. Woodle of the desirability of a more satisfactory test for workability than now exists. Admittedly, the slump test is an imperfect measure of workability. This is especially true in a comparison of mixtures of widely different nature; for example, a 1:3:6 mix with a 5-in. slump would not be at all comparable in ease of placement

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† Director, Eng. and Research Div., National Sand and Gravel Assoc., Washington, D. C.

to a 1:1.5:3 mix with the same slump. However, in the absence of a more satisfactory method, the slump test was included in this specification to mark the limits of consistency that would avoid, on the one hand, mixes too dry to puddle properly and, on the other hand, those so wet that segregation would be inevitable. The slump test finds its greatest usefulness for those combinations of fine to coarse aggregate that are more readily placeable, such as are recommended in the specification and the paper. Most of the objections developed from the use of this test have arisen where an effort has been made to hold the consistency to a very narrow range of slump, or to an exact slump, and where it has been used as a method of controlling the quantity of mixing water. However elusive that intangible quality of workability may be, lack of workability, on the other hand, is neither elusive nor intangible, but is readily recognized.

Mr. Watson has made valuable and comprehensive comments on the different clauses of the specification. While in entire agreement with many of the points he raises, the writers feel that some of the questions may have arisen in his mind through their failure to outline in more detail the scope of the paper and specification.

His comment on the differences in strengths in Fig. 1 and Table 1 for the same water-cement ratios may be explained on that basis. The two sets of data represent a span of six to eight years in the history of the laboratory and include changes in technique as well as differences in materials. The tabular values are from recent tests and are typical of the higher results now commonly being obtained, both in the laboratory and in the field. Neither set of data has been corrected for absorption of water by the aggregates. If the later data are thus corrected, as is now being recommended, the margin of difference over Abrams' original curve is considerably narrowed, so that the original curve remains a good although somewhat more conservative basis for design.

Mr. Watson points out that some of the discrepancies in the data of Table 1 can be explained by the greater absorption where larger quantities of aggregate are used; and that aggregates which have a high absorption need the stronger mortar resulting from the reduction in water-ratio due to absorption. It is felt that Mr. Watson's point is well taken in extreme cases.

With reference to the differences in strength in Table 4, these can be explained readily by the greater loss in water during moulding from the richer and, consequently, wetter mixes. For example, in Column (1) of Table 4 the 4 300-lb. concrete had a slump of 11 in.—a very wet mix, which would loose water readily—while that having a strength of 3 560 lb. had a slump of only 2.2 in.—a plastic, cohesive mix.

The writers agree with Mr. Watson that durable concrete is the ultimate aim for all structures and that in the case of concrete exposed to the elements, durability is largely dependent on impermeability. They are not familiar with the data to which Mr. Watson refers regarding the advantages

of fine sands from the viewpoint of impermeability. On the contrary, from their study of such data as are available, they are convinced that it is the proportion of uncombined water in the cement paste that determines the permeability of concrete, provided it was of the proper plasticity when placed. The evidence on this question seems quite convincing; and a logical deduction is that, in the usual nominal mixes, a coarse, well-graded sand would produce concrete of lower permeability than a finer one, due to its lower water requirements. The writers are in thorough agreement with Mr. Watson's statement that the condition of exposure of the concrete should be taken fully into account in the selection of proportions.

Mr. Watson has interpreted the clause of the specification fixing the water-cement ratio for the different strength classifications to be a specification of strength requirement. Such was not the intent; in fact, the writers pointed out that the time was not ripe for such a specification and gave in some detail the objections to it. The purpose of indicating the different classes on the basis of strength (and this was not made clear in the paper), was to meet the requirements of the Chicago Building Code, which are stated in such a way that it may be interpreted either on the basis of strength or on that of arbitrary proportions. The designation of class of concrete by letter, as Mr. Watson advocates, would have been better under other circumstances.

Mr. Watson's suggestion of narrower limits for the measurement of moisture in the aggregate is quite in accord with the writers' views. In this first specification it seemed desirable not to be too rigid and, therefore, the allowance of 2% was included. There should be no difficulty in meeting the more rigid requirement suggested by him.

He points out certain of the unsatisfactory features of the slump test, but falls into the error of assuming that it was the intent of the specification to use this test as a control of the quantity of mixing water. The slump test was used only to mark the limits of the consistency to avoid both the extremely dry and over-wet mixtures.

The section on tests of concrete to which Mr. Watson refers is admittedly not a fundamental feature of the specification. It was included to provide for the tests desired to obtain a check on this new method of controlling concrete. The results of the tests had no bearing on the provisions of the contract, except that the owners reserved the right to change the water-ratio, with proper compensation to the contractor, should the tests show this to be desirable.

Mr. Watson points out the desirability of a wider range in grading of fine aggregate than that indicated in the specification. This is undoubtedly desirable. It should be remembered, however, that the specification under discussion was written with Chicago aggregates in mind.

The additional time of mixing that he recommends would undoubtedly be desirable in many instances. Likewise, a recommendation of a minimum

temperature of 50° Fahr. during the early hardening period of concrete is an improvement over the value stated in the specification. It is believed, however, that the maximum temperature of 90° Fahr. for the concrete as it leaves the mixer is unnecessarily low. The only information in the writers' possession indicates that the limit of 120° Fahr. specified, does not approach a dangerous value.

Mr. Whitney's discussion refers principally to the difficulty of estimating quantities of materials under this specification. Usually, this is the first objection raised to it, but one that the writers feel to be less serious than may be appreciated at first thought. As was pointed out in the paper, a few simple preliminary tests with the materials to be used will give all the desired information and with much greater accuracy than the common tables of quantities used for estimating with arbitrary proportions.

Mr. Boyden raises the question as to which of the two water-ratio strength equations were used as a basis for the design. In this case the original equation of Professor Abrams, $S = \frac{14\ 000}{7^x}$, was used. Mr. Boyden is correct in stating that the slump test has been a useful device in improving the quality of concrete and that much better control is obtained by its use than by ordinary methods of procedure. It is the writers' point, however, that the best use of the slump test is in fixing the limits of consistency, and not as a means for controlling the water.

Mr. Perry's experience with the use of the water-ratio specification is interesting and, as far as the writers are aware, represents the first instance in which this method has been applied to pavement construction. His use, however, of an approximate method of correcting for the water in the aggregate on the basis of an arbitrary mix does not seem to be as satisfactory as the straightforward method of making the correction on the basis of the actual quantities of materials used. In fact, important errors may result from the use of such an approximation.

Mr. Ahlers' discussion would seem to be pointed toward a specification based on strength. The writers have pointed out the objection to a strength specification for general use in the present state of the art. It is recognized that in the hands of organizations experienced in these control methods, as is that of Mr. Ahlers, the strength specification is entirely feasible. The practice of establishing a job curve for the water-ratio-strength relation, is a distinct step toward eliminating variations due to differences in job conditions, materials, etc.

Mr. Merriman has made the point that the proper design of concrete mixtures is only one of the steps in securing the desired end—permanent construction. The writers are in full accord with this statement. The point which he makes of the importance of keeping the water within the mass during the early curing period should not be overlooked. It is the loss of water during hardening that interrupts the chemical reactions so necessary to the building up of a dense internal structure that will resist the penetration

of moisture and the beginning of disintegration. The significance of the design of concrete mixtures by the water-cement ratio goes beyond the mere fixing of a strength relation for standardized conditions of curing and testing; the quantity of mixing water is the basic factor in the production of impermeable concrete, for it is the uncombined water in the mass that fixes the degree of porosity of the concrete. This uncombined water is a function of the original quantity used in mixing and the extent to which the chemical reactions are completed.

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Paper No. 1633

UNIT STRESSES IN STRUCTURAL MATERIALS

A SYMPOSIUM*

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* Presented at the Special Meeting of the Structural Division, New York, N. Y., October 28, 1925.

UNIT STRESSES

By F. E. Turneaure,* M. Am. Soc. C. E.

The general history of the principles underlying the development of modern practice in steel design has been quite fully set forth in the report of the Society's Special Committee on Stresses in Structural Steel†—especially with reference to unit stresses. The stress of 16 000 lb. per sq. in., has a rather ancient tradition, and is based to a considerable extent on standard railroad bridge practice. It has had an undue influence on specifications for other purposes, and the conditions under which it is used have not been given sufficient consideration. The question of basic unit stresses for bridges was discussed extensively and at many sessions by the Iron and Steel Committee of the American Railway Engineering Association. Many members of the Committee favor and use a stress of 18 000 lb. per sq. in. The old unit of 16 000 lb. per sq. in. was, however, retained, but it is well known that the chief reason for this is to provide for a considerable future increase in loading beyond the design loading; that is, it was decided to adhere to the common practice of permitting an increase of load under the normal methods of operation considerably beyond the design load before replacements were needed. Bridge engineers of railway companies have been loath to give up this margin, which they retain more or less in their own control. As a measure of the real capacity of such structures, unit stresses of 22 000 to 26 000 lb. per sq. in. are used, and old structures are rated on this basis.

Furthermore, it is a not uncommon practice in the design of large structures to calculate them on the basis of a unit stress of about 24 000 lb. with a live load 50% in excess of present maximum loads. This method assumes a working stress of 24 000 lb. for dead load as well as live load, and the dead load in such cases is relatively large. The unit of 16 000 lb. per sq. in. used in bridge design, therefore, cannot be considered as directly applicable to building design, for which the conditions are very different.

In the matter of column stress, the same general condition obtains, and, in addition, a further factor has caused a considerable reduction in unit stress within the past few years. Prior to 1920, the maximum stress allowed in the specifications of the American Railway Engineering Association was 14 000 lb. per sq. in., but as a result largely of the tests‡ of the Society's Special Committee on Steel Columns and Struts, especially on thick sections, the maximum was reduced to 12 500 lb. In the rating of existing bridges, however, a maximum unit stress of 20 000 lb. per sq. in. is permitted for open-hearth steel.

On the whole, therefore, it would seem that to apply the units used in railroad bridge practice to a code for building construction, in which the

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† *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

loads are not supposed to be materially increased and for which, as a matter of fact, the design load may seldom or never be realized, must be considered as ultra-conservative, even taking into consideration the possible difference in quality of material. A specification for bridge design consciously intended to provide for a large increase in load has had, it would seem, too great weight in building design practice for which no such increase need be expected. It is true, of course, that specifications for building construction have been independently studied and developed for many years; but probably bridge engineers, owing to the many replacements caused by overloaded conditions, have had experiences in maximum practicable stress units that are not so often furnished, or so readily noted, in the field of building construction.

The development of reinforced concrete practice has had, to a considerable extent, a different history from that of steel design. Being primarily a problem of the building rather than of the railway engineer, the subject has been carefully developed and brought up to date in a report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.* This report increases several of the important working stresses recommended by the previous Joint Committee,† and proposes stresses higher than those in use by many engineers. It is the result of a long, careful consideration, and undoubtedly represents good modern practice. In some respects, however, the speaker considers the units proposed as liberal rather than conservative and of doubtful wisdom in a building code unless set forth in a somewhat different manner than that presented.

The building code proposed by the Building Code Committee of the U. S. Department of Commerce,‡ if it meets the purpose intended, will serve as a standard for consideration by States and cities in the formulation of their own codes. Such a code should represent, as far as possible, standards of good practice as to design and execution that can reasonably be expected in the community where it is to apply. It cannot provide against faults of gross ignorance, nor can it assume expert talent.

Undoubtedly, this standard will operate to the disadvantage of the best practice, but it seems to be the price necessary to pay in order to avoid too numerous failures, on the one hand, and drastic governmental control on the other. Under these conditions, building codes must necessarily be rather conservative and follow the development of the art rather than lead it. Much discussion is necessary before the advantages of the latest knowledge can be made available in building codes. It would be a misfortune for any one code to be adopted generally by all units of government. Certain it is that many cities, with high-grade building departments, first-class engineering talent, and skilled workmen, can operate under a much more liberal code than a State which must take into account construction in all kinds of places. The

* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1143.

‡ This Building Code Committee made tentative reports for private distribution in May and September, 1925, covering "Recommended Building Code Requirements for Unit Working Stresses in Building Materials." The final report is not yet (June, 1926) in print so that it is impossible to refer members to a definite publication relating to the remainder of this discussion.

problem in this case is quite different from the adoption of standard highway bridge specifications for various State Highway Departments, in which the design is directly under the State Bureaus. The speaker's experience relating directly to codes encourages him to take a rather conservative attitude, especially toward reinforced concrete design and execution.

The Building Code Committee of the U. S. Department of Commerce has clearly pointed out the importance of adequate tests and inspection of reinforced concrete work, as well as design, if the advantages of good practice are to be secured in the use of liberal unit stresses, and has emphasized the difference between inspection of reinforced concrete and that of structural steel work. Every engineer recognizes these points. Are these requirements, however, actually being met in general practice? Except for a small proportion of the most important jobs, the actual testing of the concrete used is hardly known in Wisconsin, except when ordered by the Industrial Commission because of suspicious conditions and the inspection is inadequate. Architects will promise to maintain continuous inspection, and will actually visit the work two or three times a week. Much has been learned in the past ten years as to the selection of aggregates, but practice is still far from what it should be.

In its recommendations as to the assumed strengths of concrete mixes, the Building Code Committee distinguishes between "plastic mass concrete", and "concrete mixed moderately wet", making provision for stronger concrete if the proper specified conditions are met. As a matter of fact, according to the speaker's experience, plastic mass concrete is seldom used in reinforced concrete work. "Moderately wet" would be a rather generous description of most of the work. In the experience of the Inspection Department of the Wisconsin Industrial Commission there has been reported only a single job on which the concrete could be classified under the "plastic mass" definition. It is either moderately wet or worse.

According to the Building Code Committee the allowed tensile stress in reinforcement may be 18 000 lb. per sq. in., under proper inspection. This increase from its earlier report would seem to be fully justified. There are few structural elements subjected to such low secondary or uncalculated stresses as the rods in a beam, or the lower flanges of a plate girder, and a unit stress of 18 000 lb. would appear to be conservative.

The speaker is opposed to allowing a unit shear stress of 12% of the compressive strength of the concrete. This is double the value adopted by the former Joint Committee on Concrete and Reinforced Concrete and is more than seems to be warranted by current practice in design and quality of workmanship. No doubt such values can safely be applied under favorable conditions, but to allow them merely on the basis of the design submitted and assuming only the ordinary inspection would not be good practice. It will be several years before building codes should go much beyond the present customary units. Compressive fiber stresses in beams may be relatively high and still leave an ample margin of safety against total failure, but shear stresses should be treated more conservatively. Safety against shear failures

depends on the quality of workmanship in a small part of the beam and in details that are perhaps the most troublesome in the whole structure. Present inspection methods do not warrant liberal treatment in this detail. The allowable fiber stress for beams seems rather high but this is of much less importance than the shear stress.

Referring to the unit stresses for steel recommended by the Building Code Committee, it is noted that the revised Final Report* of the Special Committee on Stresses in Structural Steel permits 18 000 lb. per sq. in. for steel which change meets the specifications of the American Society for Testing Materials. This unit is conservative and is probably as high as is warranted in a general code. It is relatively more conservative than some of the proposed concrete stresses. There is no question but what a unit of 20 000 lb. per sq. in. can safely be used under favorable conditions of design and execution and should, if possible, be provided for. The maximum stresses allowed for columns are low, especially for values of slenderness ratio less than 40. For such short columns they could well be raised at least 1 000 lb. in both cases.

The foregoing suggestions have been made from the standpoint of securing a balanced code that will be conservative and at the same time provide, as far as practicable, for work of the better class.

* *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 439.

UNIT WORKING STRESSES IN REINFORCED CONCRETE

By A. E. Lindau,* Assoc. M. Am. Soc. C. E.

The proposal of the Society's Special Committee on Stresses in Structural Steel to increase the basic working stress from 16 000 to 20 000 lb. per sq. in.† seems a radical step to many structural engineers. The time-honored basic working stress for steel structures had been accepted so long without question by the average designer that it was considered a matter of fact, rather than a question of engineering judgment. Whether he was so "inexperienced" as to believe that he had a factor of safety of four or enlightened enough to understand that the yield point rather than the ultimate strength of the material was a better basis for estimating his margin of safety, was immaterial.

He felt that no radical change had taken place in the industry. The quality of steel remained substantially unchanged; in fact, such slight changes as did take place were revisions downward of the minimum ultimate limit, which would naturally not suggest an upward revision in the working stress. He realized, however, that time had brought improvements in practice and workmanship, and substantial advances in methods of structural analysis.

The serious proposal to select a working stress with a margin of safety as low as 50% has stirred up interest in working stresses, not only in steel construction, but in other building materials. The principal function of the structural engineer is to use materials as economically as is consistent with safety and endurance. The problem of selecting working stress may properly be stripped of indeterminate factors other than those that pertain to variability in quality of material and workmanship; therefore, a critical examination of the behavior of structural material under stress is required.

The history of working stress in reinforced concrete construction does not differ greatly in general character from that of steel or other building materials. In the early stages of its development, working stresses were naturally conservative, as little or no experimental data were available regarding the resistance of concrete to various kinds of stresses. The early engineers were pioneers and blazed their way with intuition as their principal guide. Frequently they tested their factor of safety by loading sections of the completed structures to the extent of two or more times the working load. By noting the behavior of the structure, they obtained what seemed the final answer, namely, the load-carrying capacity of different types of structural members. It is true that these tests were often crude and faulty in execution; but the surprising fact remains that with few exceptions these structures have given a good account of themselves under service conditions. Hence, if based on service conditions for determining working stresses, the early work in reinforced concrete was commendable.

* Pres., Am. System of Reinforcing, Chicago, Ill.

† *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

More than twenty years ago the task of determining the resistance of concrete and reinforced concrete to stress was taken up in a comprehensive way by well-equipped laboratories under the direction of experienced investigators. Accurate strain-gauges were devised and the new technique stimulated research in many places. Beams and columns, slabs and frames, were loaded and stress intensities measured. Laboratory methods and strain-gauge readings were applied to completed structures, thus determining within limits the stresses to be expected under certain conditions of loading. In addition, extensive tests on the compression strength of concrete were undertaken. One laboratory alone has made more than 150 000 compression tests, 15 000 wear tests, 2 800 transverse bending tests, and many other tests. It is safe to say that no other structural material has been so extensively and minutely investigated.

As a result, false theories have been exploded and many bad practices eliminated. The details of design and construction have been vastly improved. It is only natural and entirely legitimate that, in view of the accumulation of information concerning the physical properties of concrete and the improvements in details of construction, working unit stress should be increased and placed on a more rational basis. An examination of the building codes of the principal cities in the United States discloses the fact that there is a tendency toward upward revision.

Unfortunately, a discussion of structures in use gives only the most general idea of the margin of safety. At best, it merely establishes the fact that the practice in use is satisfactory, and tells nothing as to the feasibility of changing working stresses nor as to the amount of such change.

In considering working stresses, importance has been attached to the fatigue of material under long-continued or repetitive loading. In concrete, the compressive strength as a basis of design is usually considered the strength at 28 days. As is well known, there is a considerable and progressive increase afterward which may compensate for the fatigue effects. Furthermore, reinforced concrete is generally built as a monolithic structure permitting the wide distribution of local loads, thereby in a measure decreasing the intensity of the stress at critical sections.

Compression.—Without question concrete under present average conditions has greater variability in quality than steel, which factor is important in the selection of working stresses. While it may be contended that the average strength of concrete is satisfactory, occasionally test specimens show abnormally low strength.

The vast amount of test data on record would no doubt yield an answer to this question of variability. The objection, however, is made on the one hand that most of these tests are done under laboratory conditions and may not be representative of field conditions, and on the other hand that the specimens are small and therefore not representative of the strength of full-sized structural members.

Perhaps the best answer to the first objection is the series of field tests* of concrete made during the summer of 1923 through the co-operation of a group

* "Field Tests of Concrete," by John G. Ahlers, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Concrete Inst., 1924, p. 358.

of New York contractors and the Structural Materials Research Laboratory of the Lewis Institute of Chicago, Ill. This series involved the comprehensive testing for quality of concrete in the construction of seven different jobs built by as many different contractors under weather conditions ranging from April to December of that year. Investigation No. II* yielded among others the following conclusions:†

"1. On these five jobs where trained organizations of long experience were used, the proportions and methods employed, based on experience only, gave concrete having an average strength from 6 to 18 per cent. greater than that assumed for purposes of design. It is noteworthy in this connection that many objectionable practices which account for unsatisfactory concrete, were not encountered on the tests.

"2. With concreting methods used on these jobs, it would be fair to expect a uniformity of strength such that not less than 75 per cent. of the test specimens would fall within 20 per cent. of the average strength, and not more than 5 per cent. would fall below a minimum value computed on the basis of the Joint Committee Tables of Proportions."

Quoting again from the field tests by W. A. Slater, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., the authors, in their "Concluding Remarks", state:‡

"The averages of the strengths found at Camden and Newark Bay were as great or greater than the strengths which the mixes might be expected to yield. They were greater also than the strengths used as a basis of the design. The average strength for the Camden 1 : 2 : 4 concrete was greater also than the corresponding strength of laboratory concrete made from materials shipped from Camden to the Bureau of Standards."

Generally not more than 10% of the specimens of any mix gave strengths less than 80% of the average strength. Laboratory tests do not usually show much greater uniformity.

The second objection, that tests of small specimens may not be representative of the strength of larger units, would seem to operate in favor of uniformity rather than otherwise, in that accidental variations of a small part of a batch—or even the abnormal strength of a particular batch—are merged with others and tend toward maintaining an average result. The service record of completed structures, and such tests as have been made on structures after completion, apparently sustain this view. In fact, strain-gauge readings and load tests—especially where the assumed static working load is applied—in general indicate stresses so much lower than those expected that they may be even misleading. This is particularly true with reference to stress measurements in the steel reinforcement.

In reinforced concrete design the compressive stress in concrete and the tensile stress in the reinforcement are the governing elements. The stresses selected, together with the ratio of the moduli of elasticity, definitely fix the amount of the reinforcement. General practice as well as the recommendations§ of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete fix the area of steel as less than 1% of the effective section of the beam. It is significant that reinforced concrete beams rarely

* *Proceedings*, Am. Concrete Inst., 1924, p. 358.
† *Loc. cit.*, p. 382.
‡ *Loc. cit.*, p. 431.
§ *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

fail in compression and then only when the reinforcement is double or more than double the amount now used. Failure is in general in the tension of the steel or in the bond between the steel and the concrete. Furthermore, the stress-strain relation in compression is not a straight line up to the ultimate strength. This has been shown by Mr. Slater to cause a shifting in the position of the neutral axis and therefore to give a greater flexural strength to the beam than may be expected from the tests of the cylindrical specimens indicating low compressive quality of the concrete.

Shear.—The shear determines the diagonal tension or web stresses in flexural members. The resistance of beams to shear is intimately bound up with the details and amount of the reinforcement. In recent specifications and codes, and particularly in the Standard Specifications of the Joint Committee, special provisions for anchorage, bent-up main reinforcing bars, and stirrups are required where high shearing stresses are permitted.

In the speaker's opinion there is sufficient test data to support the contention that the Standard Specifications of the Joint Committee yield an ample margin of safety. Dependence is placed largely on the reinforcement, so that variability in the quality of the concrete enters into the problem only to a comparatively small degree.

Tension in the Steel.—The choice of a working stress in the steel reinforcement is subject to the same process of reasoning as may be used in connection with steel structures, with this fundamental difference—that in reinforced concrete steel of a high yield point may be used. It has not the same limitations due to fabrication or in use in compressive members as has structural steel. Furthermore, the tensile strength of the concrete is entirely neglected in the design, whereas it operates to reduce materially the actual stresses, at least within the range of working conditions. Therefore, if it is agreed that present working tensile stresses in structural steel design are too conservative, the same logic would apply to the steel stress for reinforced concrete.

The discussion of unit working stresses in reinforced concrete is particularly timely. The present day is a period of great advance in the production of concrete of much better quality and particularly of greater uniformity than heretofore. Recent investigations of the underlying principles governing quality and uniformity have yielded surprising results. It is confidently predicted that by proper control of material and workmanship, concrete can be made the most uniform in quality of any building material.

Fortunately, the important factor of control is a comparatively simple one, easily applied to field conditions and needs only to be fully appreciated to be generally applied. In one of the largest concrete construction operations in recent years simple field-control methods resulted in concrete in which more than 90% of the test specimens met the expected strength.

There is still a vast amount of research work that should be done in order that concrete structures may be better designed and erected; but unless future investigations completely reverse the trend of the past, it seems probable that the unit working stresses in reinforced concrete will be revised upward.

UNIT STRESSES IN STRUCTURAL STEEL (FOR BUILDINGS)

By D. B. Steinman,* M. Am. Soc. C. E.

Recent reports and recommendations on working stresses in structural steel, and the general trend of the discussions on that subject, evidence a growing conviction that existing practice is unduly conservative.

There is always a tendency to perpetuate old standards long after they have outlived their usefulness under changing conditions as, for example, in the matter of working stresses for structural steel. The retention of a working stress that is generally recognized as unnecessarily low, is conducive to looseness in assumptions as to loading and to other objectionable departures from careful scientific design. These thoughts find emphatic corroboration in the Final Report of the Society's Special Committee on Stresses in Structural Steel.†

A re-study of current practice as regards working stress is in line with the replacement of the former unsound and misleading concept of "factor of safety" as an empirical fraction of the ultimate strength by the new concept of "margin of strength" as a more carefully estimated allowance below the yield point or service strength of the material.

The old concept of the "factor of safety" was an arbitrary blanket factor to cover all the elements of ignorance and uncertainty in the problem of structural design, such as, loading distribution, future increase in loading, impact, fatigue effects, inaccuracies of stress analysis, undetermined secondary stresses, strength and uniformity of material, and structural deterioration from neglect. The working stress of 16 000 lb. per sq. in. thus reached, has remained in official specifications and building codes, by inertia, to this day.

If designing practice is to be maintained on a logical and economical basis, in step with the general advancement in the structural art, engineers must abandon this unbalanced and wasteful procedure, and substitute instead a program of evaluating the different elements of uncertainty, and allocating the respective corrections where they belong. Any anticipated increase of load should be covered by increasing the assumed live load, not by reducing the basic unit stress. Any impact or shock effect should be covered by adding a proper dynamic increment to the live load stress, not by diminishing the basic unit stress. Structural deterioration should be minimized by proper detailing and maintenance, and, where inevitable, should be covered by a suitable increase in the thickness of metal in the affected parts, not by a general reduction in the basic unit stress. Only the uncertainties in the strength and behavior of the material, and such other elements as cannot be allocated elsewhere, should be included in the margin that determines the basic unit stress.

* Cons. Engr., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 396.

Even if there had been no improvement in the quality and strength of structural steel, in the accuracy and reliability of stress analysis, in the design of structural members and details, and in the general art of construction, the elimination and re-allocation of many of the elements of uncertainty that were covered by the old "factor of safety", would certainly justify an upward revision of the old working stress.

In bridge specifications, the low basic working stress of 16 000 lb. per sq. in. has persisted, admittedly as an indirect method of making provision for future growth of live loads. A better balanced and more efficient design would be obtained by allowing higher unit stresses; but the indirect and unscientific procedure is retained merely to avoid increasing the nominal design loading beyond that immediately in prospect.

Building practice has followed bridge practice in adopting and retaining 16 000 lb. per sq. in. as the standard basic working stress, but the same reasons do not apply. The bridge engineer has been confronted with the experience of continuously increasing weights of railway and highway loadings. His use of a low design stress of 16 000 lb. per sq. in. has enabled his bridges to stand up safely under progressive load increases of 25 to 50 per cent. Building design, however, is not generally subject to such increases, so that there is little or no justification for burdening building practice with the low working stress that is still retained, on an unscientific basis, in bridge specifications.

That bridge engineers recognize the safety of higher working stresses is evidenced by the current practice, based on the adopted recommendations of a committee of the American Railway Engineering Association, of rating existing bridges on the basis of working stresses of 26 000 lb. per sq. in. This indicates the confidence of engineers in the strength of steel when no evidence of structural deficiency is visible and when the loads are accurately known.

When not restricted by standard specifications, engineers have designed bridges with working stresses of 20 000 lb. per sq. in., or higher. Stresses ranging up to 25 000 lb. per sq. in. have been used in the design of important members of large bridges. All such structures have shown entirely satisfactory behavior in service.

Similarly, when not restricted by building laws, engineers have designed buildings for working stresses of 18 000 to 22 000 lb. per sq. in. Many older structures, built when the calculation of stresses was less comprehensive, now are subject to stresses exceeding 20 000 lb. per sq. in., but show no service weakness.

Despite these considerations, structural practice in America is generally limited to a working stress of 16 000 lb. per sq. in., the value established by rule-of-thumb a generation ago. European practice is less conservative; the standard unit stresses range from 17 000 to 18 000 lb. per sq. in., usually with steels considerably inferior to the American product.

Just as the old "factor-of-safety" concept misled the owners of buildings as to the permissible over-loading, so the present enforced use of a design stress that is known to be too low has led designers of buildings to take liberties in loading assumptions and otherwise to depart from conscientious and scientific design. Examples of objectionable and arbitrary practices that have

developed are the conventional ignoring of the dead weights of columns and column covering, and the insufficient allowances for weight of movable partitions, resulting in actual unit stresses that are much higher than the nominal 16 000 lb. per sq. in. The adoption of a higher working stress together with more conscientious and adequate loading assumptions will yield, for the same cost, a structure of assured greater efficiency and longevity as measured by the uniformity of strength margins in all its parts.

The foregoing considerations indicate the propriety and desirability of an upward revision of the governing unit stress from its present prevailing value of 16 000 lb. per sq. in. As a basis for determining a proper value for a revised working stress, the Special Committee on Stresses in Structural Steel analyzed the test reports of 6 000 heats of structural steel, including representative tests of the product of 6 mills over a period of 5 to 6 years. These tests showed a general range of yield point from 32 000 to 44 000 lb. per sq. in., with an average of 36 500 lb. The Committee conservatively suggests that working stresses be based on the low value of 30 000 lb. per sq. in. for the yield point, even if the mill tests studied indicate that a higher value might properly be used. Tests on full-sized riveted tension members generally show a yield point well above 30 000 lb.

Structural engineers are now generally agreed that the yield point is the proper datum for estimating service strength. The working stress should be determined by the margin of strength that is regarded as necessary below the yield point. In the absence of a mathematical basis for establishing this margin, recourse must be had to engineering judgment based on past experience in the structural art.

In recommending 20 000 lb. per sq. in. as a basic unit stress for future building design, the Committee left a safety margin of 50 per cent. Past experience shows that this ought to be ample to provide for abnormal loads and imperfections of material and structure. The 50% margin, as stated by the Committee, is intended to provide for chance conjunctions of unusual conditions, resistance to momentary effects, such as shocks, and common secondary stress actions which are reduced by re-adjustment through yield, and not for unintelligent calculation and designing, nor added loads or increased service.

If the dead loads are conscientiously computed, and the live loads used are the maximum anticipated during the service period, a basic unit stress of 20 000 lb. per sq. in. appears to be amply safe. A generation ago bridge engineers used Cooper's specifications, in which the prescribed working stress was 20 000 lb. per sq. in. for dead load and 10 000 lb. for live load. The 20 000-lb. working stress was accepted as safe for dead load at that time, and it is certainly a safe unit for dead load stress now. The stress that is safe for dead load should be the prescribed basic stress in a modern specification; any other procedure produces unbalanced designs. Any correction for the difference between live load and dead load effects should be made in the evaluation of live load stress and dynamic increment.

Replacement of the working stress of 16 000 lb. by a higher value will remove the justification of present loose methods of load calculation. The

advocacy of reduced live load specifications is a step in the wrong direction. With careful allowance for all dead load, and with ample specifications for the maximum anticipated live load, a unit stress higher than 16 000 lb. would certainly be justified. The combination would represent improved designing practice, and would be a correction of conditions that are retarding the development of the structural art. Low stresses do not insure against lack of skill and improper work; they induce it.

The recommendation for the increase of the prevailing working stress from 16 000 to 20 000 lb. per sq. in. is based on the premises that proper skill and care will be used in designing, detailing, and construction and the Committee is careful to prescribe the required conditions.*

With the basic unit stress of 20 000 lb. per sq. in. for direct tension, the Committee recommends 20 000 lb. for the maximum fiber stress in bending. It has been suggested that this value might properly be increased in consideration of the supporting influence of lower stressed fibers adjacent to the point of maximum stress in the extreme fiber. At any rate, with the accompanying reduction for length of unbraced compression flange, this specification of the Committee appears to be conservative.

For columns, the Committee recommends a maximum direct stress of 16 000 lb. per sq. in., corresponding to a safety margin of 60% before the minimum service strength, conservatively estimated at 25 000 to 26 000 lb., is reached. The maximum stress of 16 000 lb. is for columns having lengths less than 50 times their radii of gyration. The recommended column formula for greater length is $16\,000 - 80E$, in which, E denotes the excess of this length-ratio above 50. On the basis of past test results, this formula provides an adequate and consistent reduction of column stress in comparison with the basic unit stress of 20 000 lb. in tension.

The American Institute of Steel Construction, also recognizing the waste represented by the 16 000-lb. working stress, has adopted 18 000 lb. per sq. in. as a basic unit stress for structural steel. A number of cities likewise have adopted this value in their building codes. In conservative Great Britain, a working stress of approximately 18 000 lb. has long been the standard.

If the profession is not ready to adopt the recommendation of the Committee for a tensile working stress of 20 000 lb., it should at least take a step in that direction by effectuating an upward revision of the basic unit stress for structural steel from the old empirical value of 16 000 lb. per sq. in., to the amply conservative value of 18 000 lb. per sq. in.

* *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 403.

UNIT STRESSES IN TIMBER

By J. A. Newlin,* M. Am. Soc. C. E.

The fact that there are several hundred species of arborescent plants in the United States and a greater multiplicity of such species in the tropics has led many engineers to consider the subject of wood and its properties too indefinite and complicated to justify even a superficial study, much less a close analysis.

The speaker's purpose is to correct some misconceptions concerning wood. Under his supervision about 700 000 strength tests of wood have been performed, many of which were on structural timbers. Many others were made during the World War in connection with aeroplane construction.

People see pieces of balsa wood, or read about its being only one-sixth as heavy as water, and about other species being heavier than water. They hear that the Army and Navy are using for spruce a stress in extreme fiber in bending of 9 400 lb. per sq. in., and also that aeroplanes have been examined and evidences found that the timber in them had been subjected to a stress of 11 000 to 12 000 lb. without failure; and then, as an anti-climax, they see a table of stresses recommended by the Forest Products Laboratory, in which a "Select" grade of spruce is rated as good for 1 100 lb. per sq. in. Usually, engineers and contractors stop here and say, "Well, since the factor of safety for spruce is evidently about 12, the load indicated by a stress of 1 100 lb. may be multiplied by 4 and still be within a good margin of safety; and with this wide margin almost any species can be substituted for another in the same sizes." They do not know wood thoroughly, or they would not conceive of this margin. Once in a while wood does fail. Most of the failures occur in concrete forms and falsework. The optimistic contractor thinks he has this large factor of safety, he quadruples the load, and down goes the structure.

The engineer, as a rule, does not need to become an expert in the identification of species, nor does he have to be familiar with the properties of many species in order to use wood intelligently. It is not usually important to be able to distinguish between species that are quite alike in appearance and weight, since under these conditions their strengths would be approximately the same. As far as strength is concerned, for instance, it is not necessary to be able to distinguish between the fifty-seven different species of oak. Where resistance to decay is of importance they are divided into two groups. Again, it is not necessary—in fact, it is often impossible—to distinguish between the lumber of the various species that are classed as Southern pine. If a piece of true loblolly or short-leaf pine has the growth and appearance of typical long-leaf it has the properties of the long-leaf; or a piece of true long-leaf may be soft and light in weight and be more representative of "Arkansas soft pine"

* With U. S. Forest Service, Forest Products Laboratory, Univ. of Wisconsin, Madison, Wis.

than the average of short-leaf pine from which the "Arkansas soft pine" is selected.

The extremes of the species in general use for structural timbers, when considered from a strength standpoint, are Eastern hemlock and white fir, the weakest, and Southern yellow pine and Douglas fir, the strongest. The latter have about one and one-half times the strength and stiffness of the hemlock and white fir. Southern pine and Douglas fir furnish not only most of America's structural timbers, but also more than one-half of all its lumber.

The characteristics of the species, their inherent properties, the prevalence of certain defects and their relative likelihood to develop other defects in service, and the influence of duration of stress are all considered in assigning working stresses. Different limitations imposed on factors which are common to all species explain the difference in recommended working stresses for wood in aeroplanes and in house construction. These factors will be illustrated in the inverse order of their importance, as follows: (1) The variability of the strength of the clear wood; (2) the effects of moisture on strength; (3) the duration of stress; and (4) defects.

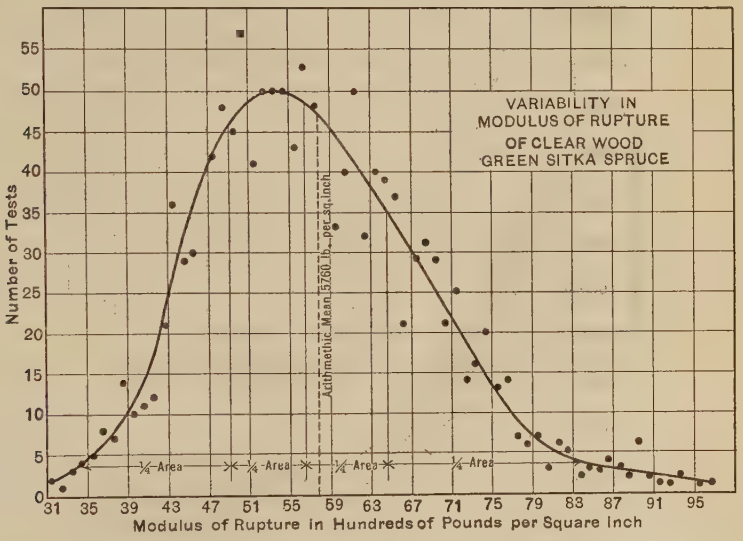

FIG. 1.

STRENGTH OF CLEAR WOOD

Fig. 1 represents the variability in modulus of rupture of Sitka spruce in a green condition. Sitka spruce is taken as an example as more data are available on this species than on practically any other, on account of its wartime importance in aircraft. The curve is characteristic of all variability curves for wood. It will be noticed that it is not symmetrical, the greater number of tests (690 out of 1 304) giving values below the average and the greater range of variation lying above the average, while the most probable value, corresponding to the highest point on the curve, is slightly below the

average. A density curve for any given species is practically a normal probability curve. The strength properties of a species vary usually as a power of the specific gravity, fractional but greater than unity, which produces an unsymmetrical curve of the type shown in Fig. 1.

In assigning working stresses, primary consideration is given to the one-fourth of the pieces lowest in strength. For aeroplanes, a lower limit for specific gravity is set which eliminates most of the lower one-tenth of the material but does not change the most probable value. In Southern yellow pine and Douglas fir it is possible to eliminate a little of this low material (along with a little of the higher) by the "ring" rule or "rate of growth" rule. A more satisfactory rule for selecting material of these species is that known as the density rule, by which density is determined from an estimate of the proportion of summer wood.

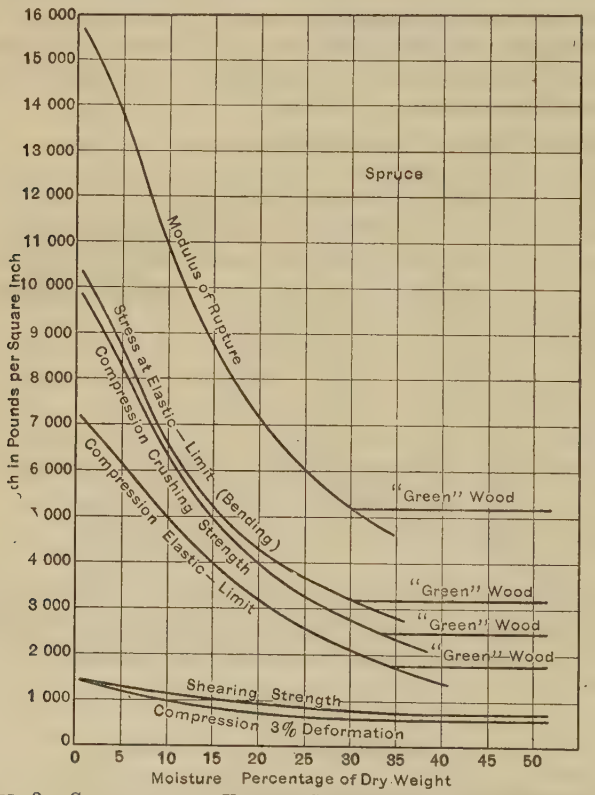

FIG. 2.—COMPARISON OF VARIOUS STRENGTH VALUES OF SPRUCE WITH VARIATION IN MOISTURE.

VARIATION OF STRENGTH WITH MOISTURE

In Fig. 2 is shown the influence of moisture on the strength of spruce wood. These are old curves. They represent work that was done quite a number of years ago on relatively few pieces of small size. More recent data show the average strength of the green material to be slightly higher than

these early tests indicate. The horizontal line at the right of each curve represents the strength of the green material; when the fiber saturation point is reached, that is, when all the free water is out of the cell, the cell wall begins to lose its water and the strength of the material goes up quite rapidly. Since in any ordinary drying some of the cells will contain free water while others are below the fiber saturation point, in practice the curve, instead of being straight to the fiber saturation point, and then having a sudden break upward, is a transition curve of varying radius, depending on numerous conditions. As between this "green" strength and the strength at 15% moisture, the most recent curve for modulus of rupture shows practically an increase of 50 per cent. The increase in the shear strength is not sufficient to be of much importance in structural material.

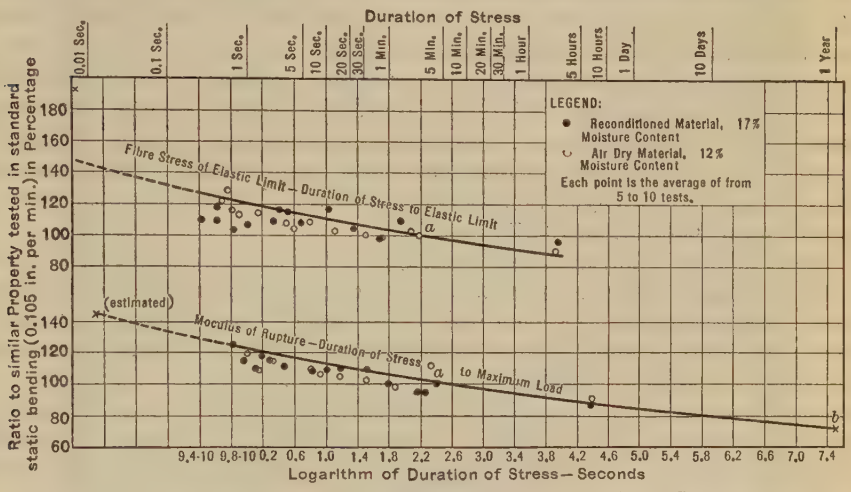

FIG. 3.—RELATION BETWEEN MECHANICAL PROPERTY OF SITKA SPRUCE AND DURATION OF STRESS.

DURATION OF STRESS

The diagram, Fig. 3, shows the influence of duration of stress on the strength of timber. A logarithmic scale is used for the time co-ordinate because it illustrates the law better than a direct plotting. The upper curve is for fiber stress at the elastic limit and the lower one for modulus of rupture. In each curve the value from the Forest Products Laboratory's standard static-bending tests, which require on the average about 5 min., is taken as 100% (point marked *a*, Fig. 3). The point farthest to the right (*b*) represents the average strength of pieces which broke in from six months to a year under dead load. No rupture strength data are available for periods of time much less than 1 sec. The point in the extreme left-hand corner (*c*)* represents an elastic limit stress in impact. In developing the curves little weight was given this point (even though it represents thousands of tests), since there are several factors which cannot be eliminated and tend to raise the point slightly.

* From *Bulletin 556*, U. S. Dept. of Agriculture.

In aeroplane flight it is impossible to maintain the conditions which give maximum stress for more than a few seconds, in which case there is a factor of almost two as compared with the load that the same material will carry for a long time—as in a library floor, for instance. This shows why timber structures should be designed for static loads and the impact due to live load neglected unless the impact stress exceeds the static stress due to the live load.

Defects

The fourth and most important factor affecting the strength of timber is its defects. One of the most serious defects is decay, which should not be allowed in any important structural member. Timber that is thoroughly air-dry and constantly kept in that condition will not decay unless it is in contact with other material that is decaying. A mill floor in Philadelphia, Pa., recently examined, revealed that wet pine had been sealed in as a sub-floor, with paper and wet concrete below and a tight top floor above, thoroughly oiled, making the conditions ideal for decay. The sub-floor rotted out completely, and the fungi passed through the top floor and base moulding into the long-leaf pine columns, rotting them out in two years. These columns in the ground, unprotected, ought to have stood for seven or eight years. Decay must be prevented in timber buildings.

Fig. 4 shows a knot, another serious type of defect in timber which often reduces the strength of a piece almost to zero. A knot is a part of a limb which, as the tree grew, became a part of the trunk. The fibers run from the bottom out into the limb. On the top they produce a kind of crinkle around the knot. Some of them seem to end there, but a number run through into the limb. The strength of the wood along the grain and across the grain is tremendously different. The variation is fifteen to fortyfold. Therefore, it would be better to have a hole the size of the knot proper than to have the knot with its attendant cross-grain.

The influence that a knot on the tension face at the point of maximum moment exerts on the strength of a green beam is approximately measured by the ratio of the diameter of the knot to the width of the face. The diameter of the knot is measured between lines parallel to the edges of the face. That is, a knot one-fourth the width of the face will reduce the strength 25 per cent. The same knot on the compression side would have about one-half this influence. Large knots have a somewhat greater influence on the strength than is indicated by this rule, owing to the relatively larger distortion of the grain around them. This effect is taken care of in the grading rules recommended by the U. S. Forest Products Laboratory so that the "Select" grade will insure 75% of the strength of the clear wood.

Large timbers never dry to as low a moisture content as small stock, and they develop more checks, especially around knots. The result is that although the average strength is increased by drying, about 25% of the timbers show no increase, and, therefore, no increase in working stress on account of seasoning is allowed. In the higher grades of joists, 4 in. and less in thickness, a slight increase in strength is recognized. In aeroplane stock the full strength

FIG. 4.—KNOT SHOWING CHARACTERISTIC DIFFERENCE IN BEHAVIOR OF FIBERS ON LOWER AND UPPER SIDES.

of clear wood at 15% moisture and duration of stress of 4 sec. may be developed.

How Working Stresses Are Determined

In arriving at a safe stress for a "Select" grade of spruce structural timber, the starting point would be 5 760 lb. per sq. in., which is the average modulus of rupture for small, clear stock tested green at standard speed. Reduce this by one-fourth to take care of the variability of the clear wood; make another reduction of one-fourth to take care of the maximum defect permitted in the grade; then take off about seven-sixteenths for the dead load effect over a number of years. This gives $5\,760 \times \frac{3}{4} \times \frac{3}{4} \times \frac{9}{16}$, or 1 820 lb. per sq. in. as the stress under long-time loading at which an occasional timber, possibly 1 in 25, would be expected to fail. Thus, the recommended stress of 1 100 lb. per sq. in. gives a factor of safety of $1\frac{2}{3}$ for this bad timber and this worst condition of loading. The average timber has a factor of safety of about $2\frac{1}{4}$ for long-time loading, and for a few minutes will show a factor of safety of 4. With average timbers under impact there would be a factor of about 6. When the influence of duration of stress on the strength of timber is considered, it is apparent why it is recommended that timber be designed for dead load and for a long period of time neglecting any impact stresses unless they exceed the stresses due to the live load which produces those stresses. Wherever tests of structural timbers are available they are found to check these factors quite closely.

For many years the Forest Products Laboratory refrained from assigning any stress to timber and it always has refused to recommend stresses where the grading rule does not limit quite closely the weakest timber that could be accepted. The stresses were finally recommended at the urgent request of engineers, and although the lumber interests were given attention, they have had no vote in fixing these final values.

DISCUSSION

LEE H. MILLER,* M. AM. SOC. C. E. (by letter).—Even if the Engineering Profession were to agree on a basic unit stress for structural steel, such agreement would constitute only the first of many steps necessary in establishing fair competitive conditions on a uniform basis. Fair competition depends on clear-cut specifications and rulings which are mutually understood by, and acceptable to, both buyers and sellers. Obviously, the establishing of a unit stress can not accomplish this condition any more than fixing the dimensions and weight of a football will assure fair competitive conditions in intercollegiate contests. Public confidence is based on a general belief that the conditions are fair, and that they are understood by all concerned, whether they be contestants or spectators.

The engineer resembles a referee or umpire, in that he assumes the obligation between buyer and seller of maintaining mutual understanding as to what is expected from each. His position is difficult even under favorable conditions, and with both buyer and seller of responsible standing. Under existing conditions, there is a wide variation in local practice, and no uniformly accepted standards. This often makes his position resemble that of a referee in equity who has no accepted precedents to guide his rulings, with the result that neither buyer nor seller is satisfied.

If the engineer is assumed to be the representative of the buyer, and not a third party functioning between buyer and seller, there are two major interests concerned in promoting better conditions for both. Responsible engineers and responsible sellers of structural steel have long felt the necessity of some step being taken, and that sooner or later this necessity would be crystallized into action.

Responsible sellers with huge capital invested in plant equipment have been confronted with financial losses due to their unwillingness to meet the constant lowering of standards which past conditions have encouraged, both in the engineering and manufacturing fields. Through the American Institute of Steel Construction these sellers have initiated, with the co-operation of prominent engineers, the promotion of a uniform specification and code of practice which together provide definitions relating to all points of interest between buyer and seller.

A committee of five men of high standing in the architectural, engineering, and academic professions, was asked to prepare a standard specification. These men have no financial connection with the sellers of structural steel. Four of them stand high in the Engineering Profession; the fifth is a senior member of a large architectural firm in the Middle West. They agreed unanimously that a unit stress without a specification requiring its use in accordance with good engineering practice, would be of no value, and they have produced what many leading engineers have approved and accepted as the best existing specification on this subject.

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Their original discussion considered a specification on a basic unit stress of 20 000 lb., but as a representative of the manufacturing interests, the writer advised them that the fabricating industry was more interested in uniform practice than high stresses, and requested that the basic stress should not exceed 18 000 lb.

In the two years following the publication of this specification, about sixty cities in the United States and Canada have permitted its use for building code regulations and many buyers have required its definitions as the basis of their purchases.

Outside the conditions fixed by this specification, there are many things on which no uniform understanding existed between buyer and seller, and which were the occasion of constant controversy. These have been exhaustively listed, and after careful analysis, rulings have been fixed in a standard code of practice, which deals exclusively with physical conditions and does not touch those which are ethical.

Definitions that are unfair to either buyer or seller cannot survive, and for this reason the co-operation of many engineers and architects was sought in developing the various rulings of the code. The correction of economic evils resembles the treatment of physical diseases in that a proper diagnosis of the conditions and causes is of primary necessity. The manufacturers who are daily receiving plans and preparing prices on work from engineers in all sections of the country, are forced into more intimate contact with the variable conditions than the engineers or architects. They have been obliged to employ expensive and efficient talent to study all the conditions and requirements of plans and specifications in order to detect unusual or difficult obligations. Manufacturing conditions are seldom the same as fixed by two engineers, and constant juggling is necessary to harmonize them with shop practice, all of which costs the buyer money or is absorbed as a loss by the seller. There are many tricks of evasion of building code requirements by both incompetent and competent engineers, neither of which observes the 16 000 lb. stress. The incompetent designer knows there is a wide margin of safety in the 16 000 lb. stress to cover his incompetence and ignorance, and sees no reason why he should employ a competent engineer when the unit stress is accepted as a proper substitute for technical training and experience. His designs include nothing that indicates consideration of unit stresses or engineering principles as the material specified is as often wasteful as it is too light.

If an increase in the basic unit stress would lead to a change in this type of design, it would be the first evidence indicating any consideration of stresses, and would have to be accomplished by turning the work over to a competent designer where it properly belongs. This result is already apparent in cities where the 18 000 lb. stress basis is used as building code requirement.

At the Annual Meeting of the Building Officials' Conference held in April, 1925, the Building Commissioner of Boston, Mass., stated that prior to the adoption of the 18 000 lb. stress, practically all plans ignored engineering principles, and that the change had resulted in almost complete elimination of

errors. Fabricators have also noticed that plans are being prepared more completely, and that competition is on a more even basis.

The evasions by competent engineers are very different in that they are based on judgment and engineering experience, and although they may not be structurally dangerous, they are unfair competitive practices which encourage a constant lowering of standards. The evasive practices used by competent designers although safe, still constitute as great if not a greater menace to fair competition than those of the incompetent because they establish precedents for more dangerous evasions. The competent engineer justifies these practices as necessary under existing conditions, as he knows the incompetent designer pays no attention to unit stresses, and the competitive competent designer indulges in similar evasions. The result is that the buyer of structural steel is in the dark as to whether he is purchasing a unit stress, a column formula, an assumption, or an omission.

The question of a technical justification for increased unit stresses has been exhaustively discussed, and it seems almost generally accepted that if properly designed, manufactured, and erected, the material may be used with safety almost to its elastic limit. Such differences of opinion as exist are based not on the ability of the material to function properly, but on the ability of the engineer to design and the manufacturer to fabricate and erect it.

It was not so long ago that a modification of the unit stress was used to take care of the difference between live and dead loads, but the Engineering Profession has practically abandoned this method of disguising its lack of knowledge regarding live load conditions. Competent engineers still express doubt as to their competitor's ability to use higher unit stresses. The answer again is not the unit stress, but the engineering principles which fix the conditions under which the unit stress will be used. The entire question is almost an emergency of the present generation brought about by the fact that there has been greater industrial and economic changes in the 35 or 40 years in which structural steel has been a commercial product than any previous 2 000 years of history.

The transition from the Iron Age to the Steel Age which was the result of temperatures that made possible the commercial production of steel ingots, started a new era, and we stand so close to the threshold that it is difficult to comprehend its significance. During such a transition the development of individualism and pioneering is natural, but the sooner everybody co-operates in the promotion of fair conditions, the quicker fair competitive conditions will be attained. This has a phase which is of National importance in that under present world conditions, National existence is dependent on ability to mobilize industries promptly more than it is on ability to mobilize man power.

To establish nationally accepted standards, needs more than the mere recommendation of nationally recognized technical organizations. It requires the aggressive effort and co-operation of responsible engineers and manufacturers to the end that the purchaser of structural steel may know that a National standard exists, which provides ample assurance that his structures will be economical and safe.

With all the practices that have come into existence under the 16 000 lb. stress, it is impossible to establish a standard on this basis, particularly when practically the entire Engineering Profession realizes that it does not provide the economy which good engineering and higher unit stresses justify. Engineers must be willing to submerge their individualism and forget that personal liberty entitles each engineer to his private column formula. Standard practice can not be attained as long as the conditions existing in connection with the 16 000 lb. stress continue; and they will continue unless they are made obsolete by the necessity of incorporating engineering principles to replace unit stresses which do not require them.

The interests of the responsible engineer are identical with those of the responsible manufacturer, and co-operation is the most effective way to attain a common objective.

E. F. KENNEY,* ESQ.—In determining the unit working stresses safely allowable in any engineering material, it is of prime importance to consider carefully the uniformity of the material and the degree of certainty with which its behavior under application of stresses can be predicted. If by computation the stresses which will be produced in an engineering structure can be predicted, and a material is available the properties of which in resisting such stresses can be predicted with certainty, the safety of such a structure is simply one of mathematics, and there is no necessity for economic waste in multiplying the so-called factor of safety. If, on the other hand, the character of the structure is such that direct and certain computation of stresses is not possible, or if the properties of the material used cannot be predicted, a much greater margin must be provided between the allowable working stresses and the ultimate strength of the material.

Structural steel is easily the most certain and uniform of engineering materials. This is due primarily to its being the result of a fusion process producing a material which up to the point where it is poured into the ingot mould is liquid. This liquid, agitated by several pourings, cannot fail to be thoroughly mixed and consequently uniform. This natural advantage is supplemented by the great advancement in the art of steel manufacture as the industry has progressed.

Those who have not been in close touch with the history of steel making have little conception of the changes and refinements which have taken place in, say, the last twenty-five years.

Twenty-five years ago practically all the chemical analyses for carbon, the outstanding element determining the strength of steel, were made by the color method. This while quick was, when compared with present combustion methods, quite inaccurate. The necessity for greater certainty has led to its being supplanted entirely by the combustion method. These improved methods helped not only in the analysis of the finished steel but made it possible to determine, with equal accuracy, the progress of the refinement in the open hearth. The old crude fracture test which in the past was the melter's

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only guide, has been supplemented by these improved analytical methods which largely remove the doubt and make the open-hearth practice almost a certainty.

Along with these strides on the chemical side, the physical testing of the steel has kept pace. Every heat is so thoroughly tested, and the physical results are so correlated with its chemical analysis, that it is regular practice to make a chemical analysis of a small test ingot cast at the time the melt is teemed, and from this analysis predict within a probable error of 1 000 or 2 000 lb., the physical strength of the plates and shapes rolled from that melt of steel. The certainty and uniformity is such that an entire melt of steel can be safely rolled into a required finished product which would probably be nothing but scrap if the predicted qualities should not be obtained.

To get uniformity in the steel, manufacturers have been driven to the closest scrutiny of the raw materials used. Everything going into the charge is analyzed and tested, and must meet closest requirements. To give an instance of the wonderful work which is being done, the record of one of the large blast furnaces at the Johnstown Plant can be cited. The pig iron was used in the manufacture of steel. All this pig iron was specified within a total range of 0.25% in silicon and had to be less than 0.05% in sulfur. The furnace was "blown in", in October, 1924, and from that date to June, 1925, a period of eight months, did not produce one cast outside the close requirements named. This is illustrative of the care with which the modern steel industry is working. These features of steel making have been briefly cited so as to show the precision which is required and attained, and the manufacture is being checked by physical testing which is growing in scope as the years go on. To some the commercial testing of structural steels appears to be just a mechanical gesture. Test after test and melt after melt is tested in the physical testing rooms. All day long at a large mill this goes on. It has gone on for years, and one hears sometimes a complaint that "none of the tests ever fails". They seldom do fail. This is the strongest proof of the certainty and uniformity of structural steel. These words "certainty" and "uniformity" are stressed because they belong peculiarly to structural steels, and they are also the words which justify a relatively high permissible unit stress in design. The present unit stress is largely due to the desire to take care of the growing loads of railroad rolling equipment. This feature should be considered in fixing unit stresses for railroad bridges, but it should not militate against the use of a proper unit stress in the economic design of other structures.

The use of yield point in determining the permissible unit stresses has been advocated. Yield point is not easily determined commercially. Assuming that the method used is the "drop of the beam", the results obtained are a function of the speed of the pulling screws, and also of the speed at which the operator moves his poise. If the method of determination is by plotting the elastic curve, it will be found that these curves are not similar, even in the same structural shape. The necessary operation of straightening affects the elastic curve. The ultimate strength is not so affected. The point of plastic yield is theoretically of interest, but the difficulties in the way of its commercial determination should be carefully considered.

G. L. Taylor,* M. Am. Soc. C. E.—Attention should be called to the fact that the report† of the Joint Committee on Specifications for Concrete and Reinforced Concrete has been incorporated in the report of the Building Code Committee of the U. S. Department of Commerce practically as a whole; but in the case of the section dealing with steel, practically a new specification has been written. This new specification does not differ materially from the specifications of the American Institute of Steel Construction, but still it is different.

Professor Turneaure called attention to the limitation of the compressive stress for columns. The recommendations of the Building Code Committee allow an increase of 25% when wind stresses are included, whereas the specifications of the American Institute of Steel Construction allow 33⅓ per cent.

One of the chief aims of the Building Code Committee is to secure standard practice, and it would appear, as Professor Turneaure also pointed out, that the stresses for steel have been made more conservative than those for reinforced concrete. It is submitted that better results would be obtained if the specifications for steel could be made absolutely identical with those recommended by the American Institute for Steel Construction, which are also in line with the recommendations of the Society's Special Committee on Stresses in Structural Steel.

Lewis D. Rights,‡ M. Am. Soc. C. E.—The speaker will confine his remarks to stresses in structural steel. The Majority and Minority Reports of the Society's Special Committee on Stresses in Structural Steel§ and the specification of the American Institute of Steel Construction have been mentioned, and Dean Turneaure has spoken of the Tentative Report of the Building Code Committee of the U. S. Department of Commerce.

The Tentative Report of the Building Code Committee issued in May, 1926, called for a basic unit working stress in tension of 16 000 lb. per sq. in., whereas the revised Tentative Report issued in September, 1926, raised this tension working stress to 18 000 lb. per sq. in.

The speaker feels that the Committee might have gone still farther and raised this tension working stress to 20 000 lb. per sq. in. which is the value recommended by the Majority Report of the Society's Special Committee on Stresses in Structural Steel.

It may be well to say a word regarding this question of a change of view, which after all seems to be a matter of education. For instance, the Special Committee on Steel Columns and Struts of the Society, of which the speaker was a member and Chairman, made a report in 1918, recommending a maximum column working stress of 12 000 lb. per sq. in.‖ This report was tentative because the work had to be closed up somewhat hastily due to the demands of the World War. A little later a Joint Conference Committee of the Society and the American Railway Engineering Association, of which

* Chf. Engr., McClintic-Marshall Co., Pittsburgh, Pa.
† *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.
‡ Vice-Pres. and Contr. Mgr., Shoemaker Bridge Co., New York, N. Y.
§ *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.
‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

Dean Turneaure was Chairman, agreed on 12 500 lb. per sq. in. as the maximum working stress for columns.

At that time engineers were somewhat influenced and perhaps disturbed by results of tests on thick materials, and it was thought well to lean on the conservative side. Since then they have gained more confidence and Dean Turneaure has stated that he would favor a maximum working stress for columns 1 000 lb. higher than that recommended by the U. S. Building Code Committee, or 15 000 lb. per sq. in.

The majority of the Society's Special Committee on Stresses in Structural Steel has gone still farther and has recommended a maximum working stress for columns of 16 000 lb. per sq. in., to correspond with a tension stress of 20 000 lb. per sq. in. The speaker is glad to endorse this change to higher stresses as a step in the right direction, which is warranted by present knowledge.

Rudolph P. Miller,* M. Am. Soc. C. E.—The Building Code Committee of the U. S. Department of Commerce, in its latest revision of the prospective report on working stresses, has, it is believed, taken into account most of the criticisms brought out in this Symposium. If some of that Committee's conclusions do not coincide altogether with those of the Engineering Profession, it must be remembered that conditions as they exist throughout the country must be taken into account. Official supervision varies considerably in quality and adequacy. In some cases, it is perhaps not at all to be trusted. Conservatism in recommendations, therefore, seems to be justified.

It might be held that the logical treatment would be to permit the use of the highest working stresses in materials of construction in order to give the public the benefit of resultant economies, on the assumption that designers are competent and honest, but it is not necessarily the best method in the interest of public safety. The recommendations of the Committee are made with ample explanations to those who are formulating codes, and it is they who must judge of what is best for their own communities.

L. S. Moisseiff,* M. Am. Soc. C. E.—Probably the speaker is not exactly on the popular side of this question. He fears that there is danger of inflating unit stresses in America as well as dollars.

There seems to be such a prevailing optimism that it is natural to think everything is lovely. Bridge engineers, however, do not have to be reminded of the state of affairs that existed about two decades ago. At that time the younger bridge engineers at least thought it was absolutely impossible for any large structure to fail, and they had a bitter lesson.

The speaker agrees, without any question, that design should be improved, that loads should be differentiated, and that secondary stresses and details should be well worked out. All engineers believe that and know that it is the method of scientific design. On the other hand, however, they should not build up an engineering Utopia.

* Cons. Engr., New York, N. Y.

For example, a theoretical case has been built up on the past behavior of steel, on the manufacture of a good grade of steel, and on a pre-supposition that good design will be used, and excellent inspection. In all steel-making that is easily possible, but not all steel construction in buildings will be competent. If it were, unit stresses should go up, and no objection could be raised. To assure this still more information on the behavior of steel should be made available, such as the behavior at the yield point, or, better, the proportional limit. It is necessary to make a study of the actual development of details and erection as carried through in the larger cities.

People who observe large buildings being erected will conclude that engineers cannot well assume the perfect conditions that the case pre-supposes. It is all very well to write and state that it is based on good design, perfect material, fine inspection, and on the integrity of every man.

There is said to be a sure way of treating a mad dog. A certain Latin formula will stop him immediately when he hears it—the only trouble is that the dog does not understand Latin. Now it may be that the details may not understand the designs. To speak practically, what are engineers really after? They do not want to build structures with more material than is considered necessary, and they want to give the public the benefit of the saving that may be obtained by raising the allowable stresses for buildings.

Incidentally, the steel manufacturers thereby might be enabled to compete successfully with the builders of concrete structures. The latter, in their turn, would try to raise their unit stresses so as to exceed steel in economy. Considered from the competitive point of view the entire attempt is founded on shifting sand, because the cost of production of steel and concrete buildings is dependent on various labor and wage conditions. For instance, an increase of $1.00 per day for the well-organized concrete union men will suffice to unbalance completely any advantage obtained. On the other hand, a raise in wages of the workers in the steel mills might swing the pendulum in the other direction.

How much economy will be effected by the projected increase in unit stresses for steel buildings? The speaker has made approximate estimates based on the cost of high buildings erected in New York City, and finds that an increase of 10% in the unit stresses of steel will result in a saving of about 1 to 1½% of the total cost of the building. Surely even such an amount when cumulative is worth saving; but from the point of view of the realty speculator who is erecting the building, it becomes a matter of secondary importance. It will not be given more attention than a variation in the necessary trim, the tiling of the toilet rooms, or the most convenient flooring. Why, then, attack the very life of the structure, its supporting skeleton, by trying to make savings that are comparatively unimportant? With more knowledge, with better control of materials, with a much better organized inspection system, these savings will be effected in due time.

It has been pointed out that Germany has raised the unit stress from its traditional 14 000 to 17 000 lb. per sq. in., or more. Americans should be happy that they do not need to do that. The United States is the richest country in

the world, and there is no reason why Americans should experience the economic strains that Germans are compelled to undergo. They do it, because they have to do it. The entire work of standardization which is being done in Germany and Russia is the result of their poverty. It is true that Americans should not waste, but they should not save where safety may be concerned.

They should build well and permanently as befits their economic standard, and engineers should show their good judgment in a better way than by trying to save 1% of the cost of a structure at the expense of its strength. These thoughts have been expressed to help illuminate the problem from another point of view and to attempt at least to moderate the strong tendency to increase the present unit stresses.

Gustav Lindenthal,* M. Am. Soc. C. E. (by letter).—The writer has held for a long time to the principle that a single unit stress should be used as a basis for dimensioning all members in a steel bridge, rather than a multiplicity of various units for various members. He also holds that the cross-section of each member should be determined for any possible combination of maximum stresses from dead load, live load, impact, wind, braking forces, centrifugal forces, and from bending and secondary stresses, the total maximum being divided by the basic unit stress to give the net cross-section of the member. The basic unit stress is taken at not less than 33% of the average ultimate strength of the steel. The method may be illustrated by citing the rules for dimensioning two heavy railroad bridges, the Hell Gate and Sciotoville Bridges, as follows:

Every bridge member shall be proportioned for that combination of maximum stresses that may occur in it simultaneously.

$$\text{Area} = \frac{D + L + I + \text{Lat.} + \text{Exc.}}{u}$$

in which,

D = dead load.

L = live load

I = impact = $\frac{L^2}{D + L} \times \frac{1\,200 + \frac{a}{n}}{600 + 4\,a}$ (a = length of train behind locomotive tender for position of maximum stress, in feet; n = number of tracks loaded for maximum stress.)

Lat. = lateral forces.

Exc. = excess = $(W + \text{Br.} + \text{Temp.}) - 20\%\ (D + L + I + \text{Lat.})$.

W = wind stresses.

Br. = braking stresses.

Temp. = temperature stresses.

u = unit stress.

The unit stress, u, for tension in the Hell Gate Bridge was 24 000 lb. per sq. in. for steel of 71 000 lb. average ultimate strength, and 20 000 lb. per sq. in. for steel averaging 66 000 lb. ultimate strength.

* Chf. Engr., North River Bridge Co., New York, N. Y.

The writer lays particular stress on the rule for impact, which is the stress about which engineers actually know the least. No steel bridge can wear out under a quiescent load; but impact can break down any structure, if it is not made strong enough to resist. It causes dynamic stresses, the intensity of which cannot be expressed in static terms, which are the only terms used in bridge calculations. The writer's impact formula is empirical, but scientifically derived* as near as may be from the well-known experience with the effect of shock on car and locomotive axles and rails. When the fiber stress (as calculated from the bending formula) is less than one-fourth the ultimate strength, these parts do not last long. If the fiber stress is decreased to one-eighth they will last longer; and if from one-ninth to one-tenth of the ultimate, they will not break at all, except through a flaw in the steel. Such behavior of steel under shock cannot be deduced by any *a priori* methods of analysis, but can be determined only from experience.

Any shock in the axle and in the rail from whatever defect in the track, is communicated, of course, to the bridge which carries the load. The impact formula is designed to follow that shock to each member and to proportion its cross-section accordingly. It applies rationally to any length of span from 3 to 3 000 ft., or more. The whole calculation is simplified on a basis as near to actual conditions as can practically be defined. The usual formula for impact in railroad bridges provides, in the writer's judgment and observation, not enough steel in the floor construction and too much in the chords; that is, a certain amount of steel is in the wrong place. Long-span railroad bridges proportioned to the usual specifications require, therefore, more steel than need be. It is good practice to proportion such important bridges for the probable maximum locomotive and car loads—using, however, for such check calculations, a fiber stress of 45% of the ultimate strength.

Robert C. Strachan,† M. Am. Soc. C. E. (by letter).—The Final Report of the Special Committee on Stresses in Structural Steel‡ has brought forcibly to the attention of the profession the matter of revising, with due regard to known facts, currently accepted practice regarding allowable unit stresses. The data cited in the report show clearly that in presuming on a high degree of uniformity in physical qualities of the material as rolled to specifications, and an average strength warranting higher unit stresses than those heretofore accepted, engineers are fully justified.

The Committee is careful, however, to include in its recommendation, the condition "that the design is made by one competent to judge intelligently and correctly, the loads to be carried and the service to be performed * * *."

Engineering may be defined broadly as ingenuity of the highest type, directed in the light of scientific knowledge, toward producing or perfecting some tangible form of aid to man's happiness or convenience. Bearing in mind also that in practical affairs the consideration of first cost, upkeep, and renewal necessarily enters, it becomes evident that the condition laid down by the Committee is of the very essence of its report.

* *Engineering News*, August 1, 1912.
† Brooklyn, N. Y.
‡ *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

The practice of modifying loading requirements, assumptions concerning the mode of application of loads, specified effect of impact, and other factors proceeds from a proper conception of the engineer's true function. Through these modifications, unit stresses differing from the "basic" may in effect be used, and such practice constitutes an important means of applying the lessons of experience.

It is not to be expected that the adoption of the recommended increases in working stresses will lead to a proportionate decrease in the weight of steel structures; for, in specifying loads, the designer will continue to use his expert knowledge of basic requirements, to the end that his critical judgment shall be satisfied with the completed construction, and that the use of "excessive or misapplied mathematics" characterized by George F. Swain, Past-President, Am. Soc. C. E., as the worst engineering mistake, may not be charged against his methods.

To sum up, the net effect of the general acceptance of the Committee's recommendations should be to make possible a better balance of design through the extension of the field governed by the engineer's judgment.

Charles S. Whitney,* M. Am. Soc. C. E. (by letter).—The writer believes that in determining safe unit stresses for structural materials the matter of building code enforcement and competency of designers should be left out of consideration. The ignorance of unskilled designers should not be allowed to prevent the efficient use of materials by competent designers. The custom of specifying low unit stresses in building codes, because incompetent designers are allowed to practice, encourages poor design and is dangerous. The unit stresses specified in municipal building codes cover a wide range, but the writer believes that there is no relation between the number of failures in the different cities and the intensities of stress allowed in their codes. Collapses are due to gross errors in design or construction.

There is another common type of failure in service resulting in cracking and disintegrating of the structure, which may be caused by the use of unsuitably high unit stresses. Any one who has designed a large variety of structures will know that the proper intensity of working stress varies considerably with the character of the structure and the service for which it is designed. This fact makes any general specification of allowable working stress unsatisfactory in its application to many particular cases.

The recently recommended increase in working stress for steel structures is logical and proper, but most emphatically the same reasoning cannot be applied to both steel and reinforced concrete structures. In the first case, the steel alone is carrying the load and any cracking of the coating due to deformation of the steelwork is superficial and does not endanger the safety of the structure. It is, of course, important in a building that the proportions of the members be such that deformations will not produce unsightly cracks in masonry and plaster, but that requirement need not prevent the use of higher unit stresses in structural steel.

* Cons. Engr., Milwaukee, Wis.

In a reinforced concrete structure, high unit stresses in the steel will cause cracking of the concrete which seriously endangers the safety and integrity of the structure. These cracks must be accompanied by the slipping of reinforcing bars permitting the entrance of moisture and the disintegration of the concrete and steel. This action is augmented by the shrinkage which occurs in all concrete after setting, increasing the cracking and deflection of concrete beams. The writer has noted several cases where the shrinkage of concrete in very shallow beams, otherwise properly designed, has caused continued deflection and cracking of masonry walls or plastered partitions through a period of two or three years after the completion of the building.

Arguments have been put forward that the actual stress in steel reinforcement is less than the computed stress and that many structures in satisfactory service were designed with high unit stresses. The effect of the tension in the concrete in reducing the steel stress decreases as the ultimate strength is approached and it therefore does not materially increase the margin of safety. It does help to reduce the cracking of the concrete under working loads.

It has been found that visible cracks occur in concrete beams when the computed stress reaches 7 000 or 8 000 lb. per sq. in. The writer has in mind two reinforced concrete structures in which beams were badly cracked. He found that these beams were not properly designed for the loading specified in the building code, but that they had never been submitted to any load which would produce a computed stress in the steel greater than 18 000 to 21 000 lb. per sq. in. Most of the load in these cases was permanent dead weight. It would appear, therefore, that those concrete structures which are designed for high stresses actually may not have been subjected to the design loads or that they have been covered so that the cracking is not apparent.

The allowable stresses for concrete structures should be determined with these considerations in mind. The cracking of concrete under its design load should not be excessive. Cracking is more serious in exposed structures than in enclosed buildings. It would also seem advisable to use a lower stress when the proportion of permanent load is high than when it is low. With a high permanent stress in concrete beams, the plastic flow of the concrete, together with the shrinkage in seasoning and the extension of the steel, may cause excessive deflection and cracking.

J. A. L. Waddell,* M. Am. Soc. C. E. (by letter).—Mr. Steinman's paper is so comprehensive, so thorough, and so sound that little indeed can be said in discussing it except to endorse the author's views. It may be serviceable, however, to emphasize certain of its salient features.

For the life of him, the writer cannot understand why specifications for structural metal do not invariably demand a minimum "commercial elastic limit" of 35 000 lb. per sq. in. for carbon steel. It is very seldom that the tensile tests of that metal show less than this; and its establishment as a standard minimum would involve no real hardship for the steel manufacturers. Years ago, the writer's bridge specifications† called for an elastic limit of 35 000 lb. per sq. in. for medium steel; and the metal manufacturers of America in those days furnished many thousands of tons of it with almost

* Cons. Engr. (Waddell & Hardesty), New York, N. Y.

† "De Pontibus," Chapter XVIII.

no protest and without making any additional charge as compared with the price of the ordinary bridge steel of commerce. There is no valid reason why they should object to doing this to-day.

As to the greatest allowable working intensities for equivalent static stresses on such steel, there may be a legitimate difference of opinion, because the true elastic limit is considerably less than the commercial one—how much less, it is difficult to say. Possibly the difference might amount to as much as 5 000 lb., thus reducing the actual limit to 30 000 lb. per sq. in.

In the old days bridge engineers were of the opinion that the intensity of tensile stress for equivalent static loads should not exceed one-half the elastic limit; and, in conformity with this opinion, the writer then specified 18 000 lb. This figure, like charity, "covered a multitude of sins," such as small eccentricities, secondary stresses (then more or less of a myth or bugbear), and lack of uniformity of metal; but, in the writer's practice, it was not intended to provide for "possible variations in load distribution," "future increase in loading," "inaccuracies of stress analysis," "structural deterioration from neglect," or that German fear-thought "fatigue of metal." Long ago the writer concluded that there is never any fatigue of metal in steel bridges, unless the actual loading, including the impact effect, be fully twice as great as the total load used in the design.

The greater accuracy of modern stress determination, the more scientific detailing of structures that has slowly been evolved, and the exclusion of most of the objectionable features of former bridge designing, make it perfectly proper to-day to fix the intensities of working stresses for bridges at a higher percentage of the elastic limit than formerly. Instead of the 50% then used, it is now perfectly safe to assume 60%, or even a little more. The better the profession is informed about the true science of bridge designing, the greater may this percentage be taken with safety. A few years hence it may be proper to adopt 65%; but, in the writer's opinion, that should be about the ultimate limit. Taking 30 000 lb. as the true (not the commercial) elastic limit, the working intensity to-day would be 18 000 lb. and that for the future, 19 500 lb., which might, perhaps, eventually be raised to 20 000 lb.

It is, of course, theoretically proper to stress steel to the same limits in bridges and in buildings, as the working intensities in both are based on equivalent static loads; but actually it is a little safer to increase stresses in buildings than in bridges, for the reasons that in the former the assumed live loads are less likely to be reached, and the impact effects are generally small and infrequent. The writer, therefore, would not be opposed to using to-day a stress of 20 000 lb. in tension for buildings, but would seriously object to more than 18 000 lb. for bridges.

In respect to the question of column formulas, the writer used to make a distinction between compression chords and all other struts. For fixed ends he adopted $18\,000 - 70\,\frac{l}{r}$ for chords, and $18\,000 - 60\,\frac{l}{r}$ for other struts. An average of these is $18\,000 - 65\,\frac{l}{r}$.

The latest proposed formula, namely, $16\,000 - 8\,E$, is equivalent to $16\,000 - 80\left(\frac{l}{r} - 50\right)$, or $20\,000 - 80\,\frac{l}{r}$. For $\frac{l}{r} = 100$, the average of the writer's old formulas gives an intensity of 11 500 lb., while the latest formula gives 12 000 lb.; hence, there is no material difference between the old and the new methods, because 100 is a fair average for the slenderness ratio.

Mr. Steinman has condensed to the limit the principles that in steel building specifications should govern the writing of the various clauses for the determination of the sectional areas of metal when he states that:

"Any anticipated increase of load should be covered by increasing the assumed live load, not by reducing the basic unit stress. Any impact or shock effect should be covered by adding a proper dynamic increment to the live load stress, not by diminishing the basic unit stress. Structural deterioration should be minimized by proper detailing and maintenance, and, where inevitable, should be covered by a suitable increase in the thickness of metal in the affected parts, not by a general reduction in the basic unit stress. Only the uncertainties in the strength and behavior of the material, and such other elements as cannot be allocated elsewhere, should be included in the margin that determines the basic unit stress."

These principles agree exactly with those which have governed the writer's practice during the last thirty years.

The weaknesses in most steel buildings lie mainly in the detailing. This is not often done with the thoroughness and the science that characterize the detailing of modern steel bridges, but certainly it should be so done. The torsion from tornadoes and the racking from earthquake shocks should be guarded against effectively in all first-class building construction.

The Engineering Profession needs an exhaustive treatise on the design of steel buildings, which would enable any competent engineer to evolve structures that would be safe under practically all conditions. It should contain specifications for both design and construction as thorough and complete as the best bridge specifications yet written; and every kind of detail that is likely to be needed in any steel building should be elaborately illustrated by drawings and explained by ample dissertation. Such a book would be invaluable to structural engineers both in the United States and abroad.

R. A. Caughey,* M. Am. Soc. C. E. (by letter).—It would seem from the papers, on "Unit Stresses in Structural Materials", as well as from similar papers and opinions, that working stresses in steel and concrete should be increased; and in view of the fact that methods of manufacture are much improved, this change would seem very logical. However, if this revision is made in any code or specification there are certain questions that should receive attention in order to safeguard structures built under such a code or specification.

First.—Specifications should be prepared more carefully. This is especially true for reinforced concrete construction for which, in many instances, the specifications are very inadequate as regards materials entering into the concrete, methods of combining these materials to make good concrete, and

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provisions for tests of the final product. These difficulties may be overcome by specifying proper qualifications and tests of individual materials, and by stating that they shall be combined in a scientific way to make concrete in which certain strengths shall be assured. The day for arbitrarily specifying a 1 : 2 : 4 mix and paying no attention to the quantity of water to be used, the consistency, the gradation of the aggregates, etc., has passed. These objections, or similar ones, can hardly be urged against steel, as steel manufacture is in the hands of experts and the tests are more adequately set forth in standard specifications.

Second.—It would seem that if working stresses are to be increased, better "field control" should be insisted upon. This would apply to both steel and concrete, and, for steel, better fabrication than is sometimes done should be insisted upon. It is hardly worth while to state that many structural members, on account of careless fabrication, are unable to carry, without a heavy dependence on the factor of safety, the loads applied to them. Getting around these difficulties would call for "field control" in the form of careful shop and erection inspection.

Concrete perhaps suffers more from lack of inspection than steel, because of the temptation to increase profits by saving cement; by wide variations in materials that are assumed by the specification writer to have no variation; and by ignorance on the part of the contractor who thinks that concrete should be mixed with water (and "lots of it"), but not with brains. The overcoming of these difficulties will call for constant inspection, not a daily visit by the architect or engineer; a constant checking of materials by tests; a changing of mix to meet changes in aggregates; and testing of the final product. These requirements may seem unreasonable and expensive, but it has been noted that attempts have been made in recent years on many large contracts to meet them by using the "water-cement-ratio" method of field control, among which may be mentioned the Portland Cement Association Building of Chicago, Ill.,* and the work of the Bronx Parkway Commission of New York City.†

Third.—An increase of working stresses should call for greater care in stress computations, notably secondary stresses. Structures have been designed in which the secondary stresses have exceeded the primary ones at certain points. A case of this kind should not be passed by without notice; and, in order that secondary and other stresses of an indeterminate nature may be more accurately calculated, field work on stress measurement as well as development of higher theory should be encouraged.

Dean Turneaure calls attention to the fact that lower values of unit stress have been used in bridge design to allow for future increases in loading. This is one way to care for such a situation; but every one will agree that if higher unit stresses are allowed, this same allowance could be made in a different and more scientific manner, namely, by an increase of loads used to compute stresses.

Mr. Steinman in a different manner, sounds the same note as Dean Turneaure, in his statement that anticipated increases in building

* See, p. 366.

† *Engineering and Contracting*, April, 1925, p. 866.

loads, effects of shock, impact, etc., should be covered by increasing the live load rather than diminishing the basic unit stress. He also calls attention to the common practice of neglecting the effects of such items as dead weight of columns, column coverings, etc., which, of course, should be considered when higher unit stresses are used.

It may appear, from the foregoing, that the writer does not favor increases in unit stresses, but such is not entirely true. However, if unit stresses are to be increased, the capacity of the materials to be stressed and the manner in which they are combined to form structural members must receive more attention. In addition, the actual stress that is carried by certain members must be more carefully determined. In other words, scientific methods must be used by all who have anything to do with construction, beginning with the specification writer and the manufacturer of the materials, and ending with the final acceptance of the structure.

There are many reasons to believe that these hopes can be realized on large projects designed by engineers who can, and will, do the work scientifically and in a painstaking manner, and constructed by contractors who have an appreciation for engineering science; but there are many places where these accomplishments cannot be expected as yet. For this reason, any upward revision of unit stresses should be undertaken only where the construction and design standard will warrant it.

The writer does not feel that steel stress should be raised to 20 000 lb. per sq. in. with yield points as at present and without a guaranty of better fabrication than is furnished on some jobs. More care should also be guaranteed with respect to distribution of steel in cross-sections of members. It would seem that, at present, 18 000 lb. per. sq. in. is a more reasonable value, and that it should be adopted only where requirements set forth in the beginning of this discussion can be met. It would also seem that 800 lb. per sq. in. is high for 2 000 lb. concrete unless the 2 000 lb. ultimate strength can be realized with almost 100% surety. It is interesting to note, in this connection, that W. F. Welsch, Division Engineer, Bronx Parkway Commission, reports* that, of thirty-two samples of concrete taken from field operations on three different jobs, only two tested below strength values for which they were designed by the "water-cement-ratio" method. This would seem to indicate that ultimate strengths can be forecasted in concrete almost as closely as in steel, if all contractors take up scientific proportioning as cheerfully as does John G. Ahlers, M. Am. Soc. C. E., who makes the following prophecy:†

"Within a few years design of concrete will be considered a simple matter. It will be this: First, the engineer will use a fixed water-cement ratio for any given strength—and get it; second, the contractor will, after fixing his water-cement ratio, be let alone and allowed to add all the sound, best-graded material he possibly can and still work his mix. Let us hope that all professional and practical people will help us bring that happy day to pass as soon as possible."

* *Engineering News-Record*, Vol. 95, 1925, p. 630.

† *Loc. cit.*, Vol. 95, October 22, 1925.

The 12% shearing stress seems excessively high; in other words, it does not seem reasonable to assume that horizontal bars with special anchorage can be depended on to increase shear resistance of beams 100 per cent.

E. G. Walker,* M. Am. Soc. C. E. (by letter).—The writer strongly supports the point of view put forward by Mr. Steinman. Safety and efficiency in design can only be achieved by analyzing the stresses developed in a structure to as close an approximation as knowledge of the subject permits, and then allowing the maximum working stress considered to be safe for the particular type of loading involved.

The argument frequently adopted, that a low working stress is taken in order to allow for uncertainties in loading, is, in the writer's opinion, weak. It is, in many cases, a cover for inadequate knowledge and disinclination to analyze designs fully, rather than the expression of a definite principle. It is unfortunately a fact that in the design of the structural framing of buildings there is a greater proportion of empirical assumption than in that of bridges. Buildings are normally designed for certain specified loads per unit area; these vary in accordance with the use to which the particular area of the building is to be put. Horizontal loading from wind is calculated at so much per unit area on certain specified surfaces with perhaps a variation of the unit pressure between the lower and the higher part of the building. As frequently as not, at the time the building is being designed, its future internal arrangements are only known in a general way. Heavy partitions and other fittings are ultimately placed in positions which are decided after the building is erected.

The only way to provide for this condition is to design the structure for such uniform load as would be adequate to cover the highest stresses developed by these unknown concentrations. It follows, however, that the actual stresses developed in floor members vary more or less considerably from the calculated stresses according to the degree in which the actual use of the building approximates to the imposition of a uniform live load. It is generally agreed that the uniform loading basis is an adequate one in that it provides for computed internal forces and moments which are at least as high as those actually developed in structures after completion, and in many cases are considerably higher.

Wind forces are treated in a highly empirical manner in civil engineering design and the calculated internal forces and moments in the various members of the structure of the building due to wind pressure must of necessity bear only a shadowy relation to the actual forces and moments developed during a wind. Investigations have shown that the distribution of air pressure over the surface of a body exposed to a uniform air current is very irregular and varies considerably with the shape of the object and its orientation to the current. In most cases the area of positive pressure is a relatively small proportion of the exposed area of the object, and the greater part of the resistance of the body is due to the existence of large areas over which negative pressures are developed. Other investigations have shown that the distribution of air pressure and velocity in an ordinary wind is extremely

* (Maxted and Knott), London, England.

irregular and uniformity cannot be relied on even between two points situated only a few feet apart. This adds considerably to the complicated nature of the distribution of pressures and suctions over the exposed surface of an actual building in a wind. In the present state of knowledge it is difficult to estimate with any degree of exactness what this distribution is, and, therefore, recourse is had to the early assumption of a uniformly distributed pressure over the surfaces out of the direct line. How approximate the forces and moments calculated on this assumption are to those actually obtaining is a matter which, the writer believes, is under investigation.

The writer has enlarged on these two points in relation to the loading of buildings in order to emphasize the argument that the computed forces acting on members of the framework of a building are at best only approximate estimates of the maximum for which the designer of the structure considers it desirable to provide. The more closely a scientific study of conditions enables a forecast of the actual loads in the members, the greater will be the liberty given to the engineer to settle the stresses on a scientific basis. It is generally agreed that where the load on a member is accurately known, proportioning it on the basis of a maximum stress of 16 000 lb. per sq. in. is unduly conservative and, therefore, wasteful of material. In the discussion of this subject, instances have been cited of computed unit stresses from 18 000 to 26 000 lb. per sq. in. in structures that are thoroughly safe. Researches on the subject also show that high unit stresses can be used with safety, so far as the strength of the material is concerned. It is, however, impossible to allow indiscriminate use of high unit stresses unless it is certain at the same time that the forces and moments which the members of a structure are designed to resist, are actually those, both in range and maximum, that the members will be called on to take when the structure is in use.

The writer's plea is, therefore, for a more exact scientific determination of the magnitude and nature of the internal forces developed in structures as the most important step required at present for the advancement of structural design. Of recent years there have been extensive investigations that have shed considerable light on the behavior of structural materials under all kinds of conditions of loading and, therefore, engineers are in a much better position at present to assess the maximum stresses which a particular material can withstand indefinitely under a particular set of conditions. Without adequate knowledge of the actual conditions of the case, however, it is unsafe to stress the material to these limits.

Thomas K. A. Hendrick,* Assoc. M. Am. Soc. C. E. (by letter).—It is suggested that a tabular compilation be prepared giving, for various structures of steel and reinforced concrete, the design unit stress, the actual field data on deformation secured by a strain-gauge, and the computed actual unit stress. A great many structures, being erected every month, should be available for first-hand data of this sort. Perhaps a great many data have already been compiled, but are not available. A thorough compilation covering all sections of the country would demonstrate the desirability of increasing or

* Civ. Engr., New Rochelle, N. Y.

decreasing the design unit stress. The new strain-gauge designed by the U. S. Bureau of Standards would be of great value for this work.

F. N. MENEFEE,* M. AM. SOC. C. E. (by letter).—In discussing the subject of unit stresses in structural materials it seems to be fundamentally correct to say that, as the theory of design approaches perfection and knowledge of the forces which come upon structures becomes more complete, it is possible to reduce the factor of safety or increase the allowable unit stresses. In addition, there is, of course, the question of the improvement in the manufacture of structural materials with reference to their strength in resisting the various forces which come upon them and the uniformity and dependability of materials.

During the past few years there seems to have been a revival of the more refined mathematical theory with reference to secondary stresses and the stresses in indeterminate members. While these studies and researches are going on, it seems timely to invite attention to the fact that there are two other phases which at present exercise a considerable control over the stresses in a structure and which are quite often overlooked. These must be carefully considered and studied to bring them up to the same standard of perfection as the theoretical portion of the subject.

The first of these has to do with the modulus of elasticity of the material. As time goes on and American steel mill practice is improved, the material turned out is, of course, more uniform and the modulus of elasticity more nearly a constant quantity; but the writer ventures to say that at present the modulus of elasticity of structural steel varies between 5 and 10%, and concrete varies as much as 50 per cent. Needless to say, refinement in structural theory will not produce greater accuracy in results than is obtained in such factors as the modulus of elasticity.

The second phase is the state of present practice with reference to structural connections. It is possible, with very careful attention to details of support and loading, to verify theory with reference to deflection and stresses in beams if the beam is long relative to its depth; but very few beams in actual practice get the same ideal or academic attention as the laboratory beam. The connections between beams and columns are of so many different kinds and standards are so silent on this subject, or so often disregarded, that the stresses in a great many structures are very different from what would be obtained theoretically. The writer has measured strains in structural steel (the most uniform building material nowadays) and has found, for instance, that the pouring of a concrete mat produced a fluid condition which in turn produced a lateral flexure of a large, heavy I-beam. This lateral flexure was not contemplated by the designers; it produced a compression in one side of the lower flange, which was supposed to be in tension. Diagonal struts, designed for the purpose of taking compression, at least when the load was static, were, in some cases, found to be in tension, showing the influence of secondary stresses due to unequal settlement of foundations or to temperature variation during the erection of the structure.

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Progress is desirable, also an increase in allowable unit stresses; but with the extremes to which structural engineers and architects are going in the matter of high buildings, with almost certain competition in their production, disregarding local foundation conditions in some cases, with the lack of accurate knowledge as to the modulus of elasticity of steel and concrete, and with the disproportionate amount of study that has been spent on structural connections, the writer is forced to take the conservative side of the question with reference to increase in unit stresses until such time as there is a better balance in the study of all the controlling phases of the structural questions.

Clement E. Chase,* M. Am. Soc. C. E.—Now that the work of the Special Committee on Stresses in Structural Steel is finished and its differences of opinion lie partly revealed and partly concealed in the wording of the Majority and Minority Reports,† there may be a feeling that enough has been said by the members of the Committee. On the other hand, an informal discussion such as this, and more particularly the contribution from the steel makers' point of view by Mr. Kenney, makes this an opportune occasion to present a few of the considerations which influenced some of the members of the Committee to oppose increasing the basic working stress to more than 18 000 lb. per sq. in.

Those who recall the Progress Report‡ of the Committee on Stresses in Structural Steel presented at the Annual Meeting of the Society in 1924 will remember that the Committee then apparently understood that as a foundation for its mission of studying the problem of the "stresses in structural steel" it was necessary first to make a survey of the material, "commercial structural steel", as it is made and furnished. That progress report summarized the study the Committee had made of mill records of the routine acceptance tests of various classes and thicknesses of structural steel. The speaker was Chairman of the Sub-Committee that went to the steel mills to gather those records and believes them to have been truly representative of mill tests as now made—with the characteristic variation in results at different mills which the experienced inspector has learned to expect.

It was part of the Sub-Committee's program, approved at the time by the Committee, to follow this preliminary survey with a series of tests on specimens cut from identical pieces of structural material. One specimen from each piece was to be put through the regular routine of testing at the steel mills and its companion specimen was to be subjected to precise laboratory testing to determine the character of the stress strain curve, in addition to checking properties revealed by the mill tests. It was thought that such a comparative study would afford a real basis of knowledge of the material involved, to serve as a starting point for an estimate of its action in full-scale members and structures.

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* Prin. Asst. Engr., Delaware River Bridge Joint Comm., Philadelphia, Pa.

† *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392, and March 1926, Papers and Discussions, p. 439.

‡ *Loc. cit.*, March, 1924, Society Affairs, p. 263.

That program was never carried out. The Committee subsequently decided that there was sufficient information already available to its members, when taken in conjunction with this preliminary survey of mill test results, fully to define structural steel as made to-day and to enable the determination of revised, up-to-date, working stresses. Much reliance was placed on the average mill test results, although in the final analysis about 6 000 or 7 000 lb. per sq. in. was "chopped off" the averages for good measure. It was assumed that a yield point of 30 000 lb. per sq. in. could be relied on, and the increased working stresses were based on this value. There appeared to be a feeling that this was a very conservative figure.

Those who have closely observed the product of the steel mill, as representatives of the purchasers, agree that as to uniformity of chemical content and ultimate strength the present-day product is highly satisfactory. As regards yield point or elastic properties it is another story. The experienced observer knows that there is no high degree of uniformity of elastic properties in the actual product and that the values for yield point reported in mill tests are not an accurate index of the quality of the steel in this regard. It is apparent from the discussion that at least one steel man of wide experience does not consider that the ordinary mill tests give information as to yield point on which any great degree of reliance should be placed. That he does not consider the yield point an important property even when carefully determined is another matter. The structural engineer dealing with static structures is thoroughly convinced of the basic importance of this property in his work. The mechanical engineer is generally interested in fatigue-resisting properties. He may conclude from recent tests that fatigue resistance can better be judged from the ultimate strength of the material than from its elastic limit or yield point.

The speaker's belief is that a series of stress-strain curves taken from fully representative samples of structural steel of moderate thickness would show the ratio of yield point to range from 55% down to 50% of the ultimate strength in a fair proportion of tests and even less in some cases—for material with an ultimate strength near the lower limit of the specifications (55 000 lb. per sq. in.) this means a real yield point strength less than 30 000 lb. per sq. in.

The report of the Society's Special Committee on Steel Columns and Struts* shows a stress-strain curve for material from the root of an 8-in., 18-lb., I-beam. This has a "useful limit point" of 27 200 lb. per sq. in., a yield point (taken as 0.1% set) of 29 000 lb. per sq. in., and an ultimate strength of 58 900 lb. per sq. in. Had the ultimate strength been 55 000 lb. per sq. in., a proportionate decrease in yield point could be expected. Marburg† reports for a standard 24-in., 80-lb., I-beam, the values, in pounds per square inch, given in Table 1.

Marburg found the elastic limit and yield point by extensometer. His values for elastic limit are really "proportional limits", the difference being too slight a degree of inelasticity to be of practical moment, but his values for

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), Plate XXXV, p. 1631.

† "Tests of Steel Beams," *Proceedings*, Am. Soc. for Testing Materials, 1909, p. 385.

yield point are pertinent. Note that the ratios of yield point to ultimate strength are, respectively, 55, 51, and 46 per cent.

TABLE 1.

	Elastic limit.	Yield point.	Ultimate strength.
Web	24 500	30 500	54 800
Flange	25 400	28 400	55 000
Root	15 200	28 100	60 100

The falling off in the ratio of yield point to ultimate strength will be even more marked as the thickness of material is increased. Tests made at Drexel Institute* showed the average yield point of specimens cut from 12-in., universal mill-plates from one heat and rolled to various thicknesses, as shown in Table 2.

TABLE 2.

Thickness, in inches.	Average yield point, in pounds per square inch.	Average ultimate strength, in pounds per square inch.	Ratio of yield point to ultimate strength.
0.355	38 800	63 720	60.5
0.475	36 880	63 770	57.8
0.755	33 420	62 150	53.8
1.000	27 380	60 850	44.9
0.995	25 700	58 600	43.8

The Majority Report of the Special Committee on Stresses in Structural Steel passed lightly over the question of thickness of material. To the speaker this has seemed a point that can not be slighted in considering working stresses; for, after all, the variations of yield point in steel of the same ultimate strength are largely due to whether the material is thick or thin. It is not considered a practical mill procedure at present to change the rolling temperature and other conditions so as to bring out a uniform "elastic ratio", that is, a uniform ratio of yield point to ultimate strength. As regards the thick material, the Majority Report merely states that for buildings all material is to be subject to the specifications of the American Society for Testing Materials, but that Bessemer steel is to be barred.

It further states that if he is using heavy material, the engineer should assure himself that all the material meets the requirements of the specifications. Any one who has had experience with testing materials under the standard specifications will know how little such an injunction takes account of actual conditions. The ordinary mill acceptance test for even the thickest flange beam or column is cut from the web. Any engineer would find it an interesting experience to go to the mill and attempt to get his test pieces cut from the thick part of the section. He would certainly be told that his con-

* *Transactions*, Am. Soc. C. E., Vol. LXIII (1909), p. 264.

tract called for steel to be tested and accepted under standard conditions, and that no one had ever heard of an acceptance test specimen being cut anywhere but from the web.

There is an increasing tendency on the part of structural engineers to use thick slabs, plates, and sections, which makes it highly important that the actual physical characteristics of such material be fully known. Thick material has such obvious advantages in many cases that its use will doubtless increase.

At present (1925) there is being undertaken by a committee under the auspices of the American Society for Testing Materials, an investigation of the yield point and other characteristics of structural material. It is hoped that this committee will be successful in making a true and accurate survey of the elastic properties of the materials that are so confidently used in building construction of ever-increasing size and complexity.

Even when a careful survey of the properties of specimens of structural steel is available only a start will have been made to acquire the knowledge of steel structures that many assume is already attained. Working stresses cannot be based on yield points of specimens of the material, unless the relation between the yield points of the specimen and of the full-sized member is known. Many have taken for granted, apparently, that there is no difference between the two, at least when considering tension members.

A study of the meager data on tension members, other than eye-bars, gives ample warning that working members may not show as high yield point strength as specimen tests. The speaker has analyzed the tension tests made for the Board of Engineers of the Quebec Bridge* and by J. E. Greiner, M. Am. Soc. C. E.† A summary of the results is given in Table 3. The tension tests made by McKibben do not report the yield point strength of the members.

Examination of the Quebec and Greiner tests indicates that it would not be safe to count on greater yield point strength for the actual structure than 47% of the ultimate strength of the specimen material. For the minimum permissible ultimate strength of 55 000 lb. per sq. in., this gives approximately 26 000 lb. per sq. in.; and the fact that a single, 2-lb. specimen tested to represent a 50-ton heat of structural steel may show an ultimate strength of 55 000 lb. per sq. in. is no assurance that all the other material in the heat has even that strength. It is a well-established fact that steel rolled from a single heat will vary 5 000 to 10 000 lb. in ultimate strength, and the test specimen may not happen to be cut from the weakest material. The fact that most of the members of a structure contain material well in excess of an ultimate strength of 55 000 lb. per sq. in. will lend no additional strength to those members that are weaker.

It is frequently assumed that the tensile working stress is not an important one as tension members do not play a large part in building construction. However, one of the most vital elements in many large buildings at the present time are the heavy trusses which form a part of the lower construction and carry the load of as many tiers as the architect or designer desires to put

* Quebec Bridge Report, Appendices XI–XII.

† *Transactions*, Am. Soc. C. E., Vol. XXXVIII (1897), p. 41.

TABLE 3.—Full-Sized Tension Tests.*

Quebec Bridge Tests.

Description of test.	Materials.		Ultimate strength of net section, in pounds per square inch.	Percentage of elongation in 200 in.	Elastic limit, net section, in pounds per square inch.	Yield point of net section, in pounds per square inch.	Ratio of yield point to ultimate strength, in pounds per square inch of materials.	Number of tests averaged.
	Yield point, in pounds per square inch.	Ultimate strength, in pounds per square inch.						
One plate, 36 in. by 5/8 in. by 26 ft. 8 in....	41 420	68 700	55 352	2.45	29 000	32 710	47.6	10
One plate, 36 in. by 5/8 in. by 26 ft. 8 in...	40 165	63 170	52 170	4.5	24 000	28 350	44.8	2
							47.1	12
Three plates, 36 in. by 5/16 in. by 26 ft. 8 in.	44 030	65 620	53 860	3.9	24 230	30 400	46.3	6
Three plates, 36 in. by 5/16 in. by 26 ft. 8 in	43 430	62 910	66 760	3.9	29 900	35 400	56.3	4
							50.3	10
Two plates, 24 in. by 1/2 in..... Four angles, 4 by 3 in. by 5/16 in.....	46 700	63 620	56 500	....	27 000	33 700	53.0	2
Two plates, 24 in by 3/8 in..... Eight angles, 4 by 3 in. by 5/16 in.....	45 630	63 550	57 700	....	26 100	32 600	51.3	2
One plate, 36 in. by 5/16 in..... Two plates, 28 in. by 5/16 in..... Four angles, 4 by 3 in. by 5/16 in.....	44 660	63 770	56 690	....	28 900	31 640	49.6	2

* Values for yield point for full-sized tests are the stresses at which the extensometers began to creep perceptibly under a steady load.

TABLE 3.—(*Continued*).

GREINER TESTS.*

Description of tests.	SPECIMEN TESTS.		FULL-SIZED TESTS.		Ratio of yield point to ultimate strength of material.
	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	
Built-up tension members, Nos. 3-10, Series B; average of sixteen tests of full-sized members, metal, ¼ in. thick	38 250	58 200	44 130	50 650	75.8
Single angle tension members connected by both legs; average of two tests; metal, ⅜ in. thick	38 850	59 350	26 633	52 261	44.9
	36 300	55 200	28 247	49 838	51.2
					Average....48.0
Members connected by one leg	38 850	59 350	22 832	44 392	38.5
	36 300	55 200	24 183	42 042	43.8
					Average....41.2

* Elastic limit determined by scaling of specimen, that is, what would now be called yield point.

above them. In those trusses one must admit that the tension member occupies as vital a place as it does in any railroad bridge; and upon what full-sized tests are the designs of such members and trusses based?

The speaker feels that there is a reason for the designer of steel structures to be rather humble in the presence of those of his colleagues who specialize in concrete and have for guidance the many tests in reinforced concrete members and structures. The steel designer must admit that hundreds of thousands of tons of steel are used annually in steel buildings with practically no information at all from tests as to the strength of the completed structures or many of the details and members used so freely in their construction. The little information that exists on the strength of large sized steel members seems to indicate need of caution before making large inroads into the unknown margin of safety. In 1880 Bouscaren spoke of the "irrepressible 'factor of safety' invented to cover two large fields, the 'incidental' and the 'unknown', the limits of which should gradually contract under the light of well observed facts." That they have contracted is undeniable. For the designer of buildings to assume that there is no longer any element of the "unknown" in his materials and structures is again to invite the disasters that in the past have served to chasten the designer of bridges.

JOHN TUCKER, JR., ESQ.* (by letter).—Mr. Steinman admits the absence of a "mathematical basis" for working stress calculations, and the Special Committee on Stresses in Structural Steel admits† being unable "to discover a method of rational calculation" of working stresses. That the method of obtaining working stress values is unsatisfactory may be gained from the discordant values of 12 000, 14 000, 15 000, and 16 000 lb. for maximum permissible steel column stresses, recommended by four bodies of eminent engineers of the United States within a period of five years, without substantially different data as bases. A rational method must and can be developed. Mathematical statistics affords the method of analysis, and has been suggested.‡ All that is now required is the perfection of details of application, and the accumulation of relevant data for analysis by this method.

Mr. Steinman starts with the long current practice of "16 000 lb. per sq. in., the value established by rule-of-thumb", citing instances in which this value has been increased to as much as 25 000 lb. per sq. in. in design, and 26 000 lb. per sq. in. in rating existing bridges. In substance, he contends that, in the light of experience—certainly a dependable guide—the stresses should be increased. Thus far the exposition is faultless, but the exact amount of the increase is still a matter for conjecture. If a maximum of 26 000 lb. per sq. in. has been safely used, then why may not this value be accepted as the standard? The majority of engineers will not be too conservative to desire an increase in the 16 000 lb. value, but undoubtedly the vast majority would be against setting the figure at 26 000 lb. per sq. in. The determination of questions by majority vote may be politically sound, but it is out of place in the field of engineering.

* Washington, D. C.

† *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 396.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 1126 and 1181.

The 50% margin of safety quoted by Mr. Steinman and also by the Special Committee on Steel Columns and Struts is, of course, perfectly accidental, and without any rational foundation. This value is a by-product, the result of the difference between what is considered by good engineering judgment to be the safe stress of the material and the strength of the material to be reasonably expected. To be consistent with his desire to eliminate extraneous variables, Mr. Steinman should eliminate this 50% margin of safety. The loading of the structure would then have to be computed so that at the very maximum it would produce stresses in the steel of the exact magnitude of the safe working stresses.

This procedure is especially desirable for buildings, where the comparatively large dead load is known with considerable precision, where the maximum live load can be computed with satisfaction, and where the wind load system can be liberally designed for emergencies of extremely high winds. The use of this suggested stress without a margin of safety will not require more accurate computations nor tend to a less safe structure. If it is not economical to determine the loading with precision, then it should be assumed large enough to cover any possible case. All uncertainties are thrown into the magnitude of the total superimposed loading, and the degree of refinement desired in the design will fix the accuracy with which the actual maximum loading must be predetermined.

A stress for compression lower than for tension does not appear logical. Examination of all tension and compression coupons reported by the Special Committee on Steel Columns and Struts shows the average of the tension and of the compression "useful limit" (yield point values not given) to differ only 1 per cent. While it is true that tension specimens with drilled or punched holes will have a greater strength on the net section through the hole, the difference is not sufficient to warrant any such difference in stress as that indicated by the various committees.

Safe stress analysis based on the mill test values for yield point, is weak and most insecure, as those familiar with such test results will concede. The usual mill test is run with such high speed and such inaccuracy that it is not to be accepted as the basis of engineering design.

Experience is a firm foundation; it is an exceedingly long, tedious, and costly process; nevertheless, it must be partly relied upon, for the experimental data on which to base analyses are incomplete, and, in some instances, misleading. Thus the results of rotating-beam endurance tests on steel of approximately structural grade appear in part to be devoid of significance, for they indicate* that several million reversals of stress well beyond the yield point will be withstood before failure, whereas it is known positively that one application of the yield point stress will cause the failure of a column.

The endurance limit of structural steel elements is an unknown quantity. For steel under one loading the influence of holes and discontinuities happily appears to be of a nature to increase the ultimate strength of the material in the net cross-section, due to the pronounced ductility of structural

* *Bulletin No. 124*, Univ. of Illinois Eng. Experiment Station, p. 77.

steel at the yield point and beyond. Endurance tests on flat specimens with an included hole show* that the material in the net section through the hole has the same endurance limit as similar material without a hole. In the rotating beam specimen with the hole perpendicular to the axis, the maximum stress computed by the usual beam formula for the net section through the hole gives a value equal to the endurance stress computed from the specimen without the hole. In general, discontinuities, if they have any effect, appear to increase the strength.

The ultimate stress under endurance tests, and the effect of high local stresses at holes and other discontinuities, both indispensable factors in working stress determinations, were not broached in the Symposium. Experience with existing steel structures must be relied on to fill the unknowns in the endurance tests, and from it the yield point must be selected as the basis for safe stress determinations.

The elimination of all variables external to the structural element, more especially those of uncertainty, as Mr. Steinman strongly recommends, is even more strongly urged by the writer. It is unquestionably the first, and a most important, step to take in the development of a rational method for the analysis of working stresses. Closely allied with this attitude is the elimination of stresses to be permitted by specifications as indefinite as "plastic mass concrete" which have been criticized by Dean Turneaure.

The rational method of working stress determination should result in a formula expressing the working stress in terms of coupon test results. It is only reasonable to assume that the requirements for structural steel will not remain constant, but probably will be slowly increased. As an example, results of tests have shown† that very heavy H-sections (14-in., 287½-lb.) can be fabricated with ultimate compressive strengths of 38 000 to 44 000 lb. per sq. in., whereas previous tests on heavy 8-in., 91-lb. H-sections developed only 24 000 lb. per sq. in., and 14-in., 287½-lb. H-sections only 24 000 to 28 000 lb. per sq. in. Surely this increase in strength is deserving a concomitant increase in working stress.

Table 4 shows the mean strengths of the several types of columns, together with the coefficients of variation and skew, statistical parameters of value in determining the reliable strength, and the computed safe stress for each column type. The safe stresses are computed by the method given by the writer in his paper entitled "Reinforced Concrete Columns".‡ There being several different column lengths, the coefficients could not be computed from the mean of the group of like columns, and the variation from the best representative "length-strength" line was used as the datum for computing the individual variations.

It is not practical to use a different safe stress value for each different type of column, nor is it plausible, in view of the considerable difference in the coefficients of variation for the different types of columns, computed from

* *Bulletin No. 152*, Univ. of Illinois Eng. Experiment Station, p. 33.
† *Technical Paper 328*, U. S. Bureau of Standards, p. 68.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1074.

a number that should be sufficient to give a much smaller variation, to assume that duplicate tests and computations on the groups of nine columns would give identical safe stresses.

TABLE 4.—Steel Column Ultimate Strengths and Safe Stresses, Together with Dispersion Parameters

Type.	Column No.*	Number of columns.	Area (approximate), in square inches.	Coefficient of skew.	Coefficient of variation.	Ultimate column stress,† in pounds per square inch.	Computed safe stress, in pounds per square inch.†	
8-in. H-beam...	32 lb	9	9.15	+0.51	3.5	37 750	30 800	
	34 lb.	9	10.0	+0.09	1.9	33 660	30 400	
	62 lb.	9	17.8	−0.36	2.8	35 300	30 100	
	91 lb.	9	27.4	+0.67	4.2	25 300	19 700	
Mean	.....	36	.....	+0.23	3.1	.....	.....	
Light sections.	1	9	11.25	+0.39	2.5	33 400	29 000	25 200‡
	2	9	10.0	+0.45	4.1	34 400	27 000	
	3	9	8.7	−0.11	2.5	34 700	30 100	
	4	9	11.7	−0.55	3.0	37 300	31 400	
	6	9	13.7	−0.63	2.6	31 800	27 500	
	8	9	11.1	−0.03	2.5	35 500	30 900	
	10	9	10.8	± 0	2.5	34 900	30 300	
Mean	.....	63	.....	−0.07	2.8	34 570	29 500	
Heavy sections	1A	9	22.2	−0.50	2.1	29 900	26 600	20 750‡
	2A	9	17.0	+0.68	2.2	32 800	29 000	
	3A	9	16.4	+1.06	3.8	30 400	24 300	
	4A	9	16.9	+0.75	2.8	28 900	24 800	
	6A	9	23.7	+0.50	8.0	30 300	22 400	
	8A	9	25.7	−1.70	6.1	32 500	22 200	
	10A	9	18.6	−0.02	2.2	31 300	27 700	
Mean	.....	63	.....	+0.11	3.5	30 870	25 300	
Univ. of Illinois, *Bulletin No. 56*..	.....	10	13.0	+0.22	3.0	28 890	24 300	

* Designation in original paper, *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

† For $\frac{l}{r} = 50$.

‡ For method of computation, see text.

There is no one unique method to pursue in determining the safe stress from the group of columns. From the group of safe stresses for each type of column, the reliable value of 9 400 lb. per sq. in. for a slenderness ratio of 50 was determined. This result is interpreted to mean that if an infinite variety of column types were possible, then no one selected at random would have a safe stress of less than 9 400 lb. per sq. in.

However, there are not an unlimited, but an indefinite, number of types. Rather than attempt to define the limiting number, it was decided to pursue the method that follows. The safe stresses for the light and heavy types of column have been determined from the average ultimate column strength (for the seven sub-groups) and from the average coefficient of variation of the seven sub-types. These results, given in Table 4, show, in agreement with the ultimate strength, the superiority of the light, over the heavy, column.

It is to be noted that the steel in all columns passed the specifications of a 30 000-lb. yield point. It is unfortunate that the characteristics of the steel from which the columns were fabricated was not determined with precision from test coupons, for the stresses just determined could then be used as a criterion in conjunction with tests of future coupons to determine the stresses to be allowed in the steel from which the coupons were taken.

Concrete.—While the Building Code Committee of the U. S. Department of Commerce* specifies concrete strengths on the classification basis of the designations "plastic mass concrete" and "concrete mixed moderately wet", and the Committee gives the slump limits for these classifications, Dean Turneaure's criticism in this connection seems to be fully justified, especially as these slump limits appear more in the nature of suggestions, and not of rigorous restrictions. The Code is also inconsistent, giving for "plastic mass concrete" a "slump of 1 in. to 3 in." on page 5, and "not more than 3 to 5 in", on page 18. The writer would go further and eliminate the "very wet concrete" (slump of 10 in., or more), as a slump of not more than 8 in. will give a concrete of sufficient workability to be applied for any purpose. If this slump without the addition of admixtures is not productive of a concrete of sufficient workability, the engineer should resort to the use of admixtures for this purpose.

Little can be added to Mr. Lindau's excellent presentation other than additional data. The strength variation is undoubtedly the most important single factor influencing the value of the working stress for concrete. This variation, all factors influencing the strength being kept constant, is a function of the size of the specimen; the larger the specimen, the smaller the coefficient of variation. The phenomenon has been theoretically predicted and experimentally verified, and is of unquestioned importance in design. This law of the variation apparently does not hold for the built-up steel column, for reasons not yet wholly known.

The statistical analysis of test beams will undoubtedly reveal a considerable possible increase in the steel tension reinforcement, as indicated by Mr. Lindau. His contention that the "present working tensile stresses * * * are too conservative", due partly to the neglect of the tensile strength of the concrete, must be tempered with an important qualification. In the critical section of a simple beam the tension concrete cracks at a stress in the steel of approximately 3 600 to 4 500 lb. per sq. in., and in the region of maximum moment the portion of the load borne by the tension concrete at the working load on the beam, is negligible. Mr. Lindau's theorem is valid, of course, when applied to the Theorem of Three Moments up to the loading that causes the first tension crack in the concrete at any point. In this connection the assistance of the concrete in resisting tensile stresses becomes of importance, since by increasing the moment of inertia of beams and slabs at those portions that are not cracked, it causes the deflections and stresses to be appreciably less than those based on computations that neglect the tensile resistance

* "Recommended Building Code Requirements for Working Stresses in Building Materials," released December 20, 1926.

of the concrete.* The cracking of the concrete produces a discontinuity in the moment of inertia of the cross-section making the problem exceedingly difficult of a mathematical solution. It would appear that this subject is worthy of considerable further analytical study.

Timber.—Mr. Newlin considers all factors concerning working stresses but pursues none of them to a logical conclusion. From the data on Sitka spruce a safe stress may be computed, in place of the method used by him in reducing the value by one-quarter to cover the variation in strength of the clear wood, a procedure with apparently no foundation whatever. The frequency curve of Fig. 1 is of a complicated nature, being a compound curve not to be classified in any of the Pearson types. A graphical approximation by extrapolation indicates a reliable stress of 2 400 lb. per sq. in. Reducing this value by nine-sixteenths and three-fourths, Mr. Newlin's fatigue and defect constants, the working stress value of 1 010 lb. per sq. in. results, comparing with the recommended value of 1 100 lb. and the computed value of 1 820 lb. given by him. It is to be noted that the value of 1 010 lb. just derived is applicable to the specimens of the size used in the tests from which the value was derived.

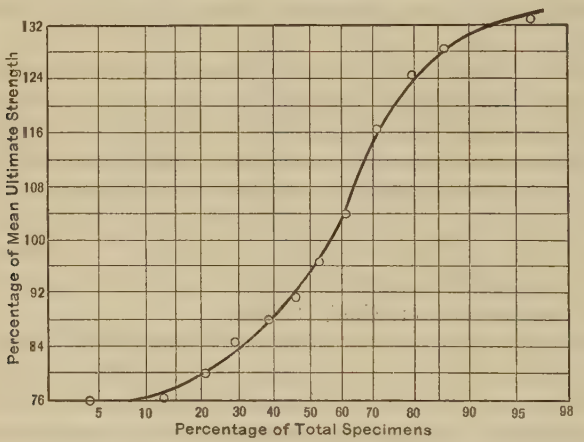

FIG. 5.—TOTAL FREQUENCY CURVE—DOUGLAS FIR BEAM STRENGTHS.

The most vital factor has been entirely neglected, namely, the difference of the strength variation with the size of the specimen. Whereas the variation apparently follows a definite law for concrete, and is independent of the size of the specimen in built-up structural steel columns, it is considerably more complicated in timber beams. This is partly due to the tendency of the timber of the larger sizes to check along the neutral axis, the locus of the maximum shear. This tendency works toward the formation of two superimposed beams each one-half the depth of the original beam. The strengths of large beams will tend to group themselves about two values—the strength of the beam as a whole, and the strength of the same beam when divided into

* *Proceedings*, Am. Concrete Inst., Vol. 17 (1921), p. 415.

two parts along the neutral axis. This latter strength will be one-half the former.

This phenomenon is indicated in Fig. 5 giving the total frequency curve for twelve specimens of Douglas fir beams that had been in service eleven years.* To compare with the test results on the large beams, forty-eight 3 by 3 by 40-in. beams cut from the larger beams after test show a practically straight frequency curve, or a normal error distribution of the ultimate strengths. These examples are given to show the possibilities of an extensive and comparatively complete analysis on sufficient test specimens.

F. E. TURNEAURE,† M. AM. SOC. C. E. (by letter).—The discussion shows that there is not a wide divergence of opinion in regard to the subject of working stresses for steel. There is also evident a feeling that accuracy and reliability in design have increased considerably in recent years, but that specifications should still be reasonably conservative.

The writer can hardly agree with Mr. Whitney that the subject of code enforcement should not be considered in preparing specifications as general as those under discussion. Officers responsible for code enforcement are, as a matter of fact, co-operating in the manufacture of a product, particularly in the case of reinforced concrete. The final product in the building is dependent, to a considerable extent, on the degree of supervision and inspection; and it is this final product that must be considered in laying down rules for design. In the case of a steel structure, the effect of good or poor inspection is not so great as in the case of reinforced concrete, and it is for this reason particularly that the writer is inclined to be more conservative in regard to the latter material.

Mr. Tucker proposes to apply the theory of probabilities to a considerable extent in determining strength values. This method of analysis may be useful where the variations in values are wholly due to uncontrollable variations in the product, and where some law could possibly be established that would represent probable deviation from the mean; but in such a structure, for example, a column, the writer believes that there is still a long way to go before exhausting the possibilities of separating the variables and reducing the controllable variations. To attempt to apply the theory of probabilities to such a complex case would seem to be misleading and without much value.

D. B. STEINMAN,‡ M. AM. SOC. C. E. (by letter).—In reviewing the discussions of the paper on "Unit Stresses in Structural Steel (for Buildings)", the writer has been pleased to note the practically unanimous agreement with his thesis that a working stress higher than the old value of 16 000 lb. per sq. in. is now justified. Apparently, the only difference of opinion is the amount of increase advisable for immediate recommendation and official authorization. Practically no one hesitates to go to a working stress of 18 000 lb. per sq. in., and some are ready to go all the way to 20 000 lb. as advocated by the writer.

Some of the discussions emphasize the fact that there still remain unknown elements affecting the safety of a structure: Variations in the material are

* *Bulletin No. 41*, Univ. of Illinois Eng. Experiment Station.

† Cons. Engr.; Dean, Coll. of Mechanics and Eng., Univ. of Wisconsin, Madison, Wis.

‡ Cons. Engr., New York, N. Y.

pointed out; imperfect designing is mentioned; and the desirability of further tests and research is suggested. The writer concedes these limitations without altering his conclusions. If these variations and deficiencies did not obtain, a working stress close to the yield point would be justified. If the technique of design were perfect, and if knowledge of loads, materials, and structural behavior were complete, the writer would not stop at advocating a unit stress of 20 000 lb. per sq. in.; under such conditions, an ultimate working stress of 30 000 lb. per sq. in. could be used with justification.

The essential point of the argument is that, although all the unknowns have not been eliminated, they have been substantially reduced with the improvements in structural art and theory during the past generation. This reduction in the unknown and variable factors justifies a reduction in the corresponding margin of safety between working stress and elastic limit. If 16 000 lb. per sq. in. was a safe working stress a generation ago—and experience has so proved it—a higher unit stress is safe to-day. If some hesitate to make the immediate advance to 20 000 lb. per sq. in., the working stress of 18 000 lb. per sq. in. acceptable to the more conservative should be adopted for the present. It is a step in the right direction.

J. A. Newlin,* M. Am. Soc. C. E. (by letter).—In discussing timber stresses, Mr. Tucker states that "the most vital factor has been entirely neglected" that is, the checking of large timbers along the neutral axis and the likelihood of failure by shear. Although the title might indicate otherwise the paper aimed to explain the method used in arriving at a safe stress in the extreme fiber in bending only. It is true that this other phase of the subject is very important.

The U. S. Forest Products Laboratory has investigated the shearing in large beams and its relation to the stress developed in the standard shear specimens, and has also made a careful study of columns both large (up to 12 by 12 in. by 24 ft.) and small. It is almost inevitable that large beams will check along the neutral axis. The checking usually increases after the timber is in place. There are, however, compensating factors, that the worst checking usually occurs in straight-grained beams having a high quality of clear wood, and that the two beams resulting from a horizontal shear failure will almost never be seriously checked along their neutral axes so that when the original beam has failed in shear at a dangerously low load it will generally be found that the two resulting beams will carry a load considerably in excess of that which caused the original failure. These two beams are, as a rule, amply strong to carry the design load safely for long periods, but they lack materially in stiffness.

Failure in shear is, however, a common cause for the removal of railway bridge stringers and also of considerable worry and expense. It does not seem to be feasible to limit shakes and checks in large timbers as closely as knots or cross-grain; therefore, to obtain a safe stress from tests of small clear specimens it is necessary to use a factor one and one-half times as great for shear as for stress in the extreme fiber.

* With U. S. Forest Service, Forest Products Laboratory, Univ. of Wisconsin, Madison, Wis.

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Paper No. 1634

SOME PROJECTS FOR SEWAGE TREATMENT UNDER THE ILLINOIS SANITARY DISTRICT ACT OF 1917*

By Samuel A. Greeley,† M. Am. Soc. C. E.

Synopsis

This paper describes briefly some of the developments under the Illinois Sanitary District Act of 1917. At this time (May, 1926), fourteen sanitary districts have been organized with populations ranging from 1 700 to 60 000. Several others are in contemplation. Three of the districts have placed sewage treatment works in operation; in five others, major construction work is under way; and, in another, work is expected to start during the year.

The works in these districts comprise intercepting sewers, pumping stations, settling tanks, and sprinkling filters. In the planning of the works, sewage has been gauged and analyzed, testing stations have been operated, and much interesting experience has been gained through investigation, design, construction, and operation. A variety of conditions have been met, including the treatment of both domestic and industrial sewages.

Costs of construction and operation are of record. Most of these sanitary districts are on streams having little flow during dry seasons so that relatively complete sewage treatment has been required. Such data as are considered useful are recorded herein.

Sanitary District Acts in Illinois

The Sanitary District Act of general application in Illinois was passed by the Legislature and approved by the Governor in 1917 and is designated as the 1917 Act. It was amended in 1919 and 1923.

* Published in October, 1926, *Proceedings*.
† (Pearse, Greeley & Hansen), Chicago, Ill.

In addition to the general Act of 1917, there are two other Acts under each of which only one district can be formed, so that they have no further applicability.

The Legislature of 1925 passed the so-called River Conservancy Act making provision for sewage disposal, development of water supplies, and policing. In order to provide a proper transfer of sewerage works from existing authorities (including sanitary districts and municipalities) to the Conservancy District, provisions are contained in the law whereby such transfer can be made upon favorable popular vote so as to compensate existing authorities for such works as have been constructed and such expenses as have been incurred.

The Sanitary District of Chicago

The Act creating the Sanitary District of Chicago, passed in 1889, has been amended several times. These amendments have extended the powers of the District and have overcome various defects as they developed through administrative experience. One amendment in 1917 provided specifically for the building of sewage treatment works. Sizable annexations have been made from time to time so that at present (1926) the District comprises more than 437 sq. miles, including 50 municipalities.

For operating expenses and fixed charges, the District may levy taxes not to exceed 0.67% of the assessed valuation. The minimum tax levy is not less than $0.18 on $100, for corporate purposes. Since its organization this District, up to 1926, has expended more than $114 000 000 for the construction of the main drainage canal and all the various connecting channels, intercepting sewers, and sewage treatment works. Of this, more than $36 000 000 has been for sewage treatment.

North Shore Sanitary District

The Act of 1911 provides that "whenever any area of continuous territory within the limits of a single county shall contain two or more incorporated cities, towns, or villages owning and operating * * * and procuring a supply of water from Lake Michigan * * * the same may be incorporated as a sanitary district under this Act".

The only area within the State of Illinois which meets this requirement is the strip of land along Lake Michigan, extending from the northern limits of the Sanitary District of Chicago to the Wisconsin State line. A part of this area was organized in 1914 as the North Shore Sanitary District. It has since been extended to comprise all the lake front from the Chicago Sanitary District to the State line. No other sanitary district, therefore, can be organized under the 1911 Act. The salient features of this Act are briefly summarized as follows:

(*a*) Three hundred legal voters may petition the County Judge to submit the proposition of organizing a sanitary district. The County Judge with two judges of the Circuit Court shall constitute a board of commissioners with authority to consider and fix the boundaries of the proposed district to be voted upon.

(*b*) If the district is approved by the voters, its management is vested in a board of five trustees who are appointed by the County Judge and the two Circuit Judges. This commission of judges divides the district into five wards and appoints one trustee for each ward. Each of the trustees is paid $500 per year and they elect a president from among their own number, who receives $1 000 per year. The trustees appoint a chief engineer, superintendent, attorney, and other employees.

(*c*) The duties of the Board of Trustees are to provide for the disposal of the sewage of the district and to preserve the water supplied to its inhabitants from contamination.

(*d*) With popular approval the district is authorized to borrow money and to issue bonds not to exceed 5% of the valuation of taxable property in the district. As this indebtedness is incurred the Trustees provide for a direct annual tax sufficient to pay the interest and to discharge the principal of the indebtedness within at least twenty years. In addition, the Board of Trustees may collect taxes not to exceed 0.33% of the value of the taxable property.

Up to 1926, this District had issued no bonds, but had built all its works, which had cost upward of $350 000, from accumulated taxes. An amendment was sought providing for the right to construct works by special assessment in general accordance with the Act of 1897 concerning local improvements, but this amendment failed. It is interesting to note that this Board of Trustees is authorized to enter into contract with any city or village for the disposal of garbage.

Act of 1917

The first district organized under the Act of 1917, in Decatur, Ill., was approved by popular election in August, 1917. Since then thirteen other districts have been similarly organized. Including the Sanitary District of Chicago and the North Shore Sanitary District, there are, therefore, fifteen districts in the State (Fig. 1).* The salient features of this Act are briefly summarized as follows:

(*a*) Whenever contiguous territory containing one or more incorporated communities shall be so situated that the construction of a plant or plants for sewage treatment will conduce to the preservation of the public health, this territory may be incorporated as a sanitary district. No territory located more than three miles from the limits of a city, town, or village can be included under such a district.

(*b*) One hundred legal voters may petition the County Judge of the County in which the major portion of the proposed district is located to submit the proposition of organizing a sanitary district. The County Judge and the two Judges of the Circuit Court consider and fix the boundaries of the proposed district, after giving opportunity for discussion in public meeting.

(*c*) If the district is approved by the voters its management is vested in a board of three trustees, appointed by the County Judge and residents of the

* Another Sanitary District has since been organized at Rockford, Ill., and engineering studies are under way.

district. These trustees elect one of their number as president and appoint the necessary officers. The salaries of the trustees are limited to $100 per year each.

(*d*) The duties of the Board of Trustees are to provide for the disposal of the sewage of the District and to preserve the water supplied to its inhabitants from contamination.

(*e*) The District is authorized with popular approval to borrow money and to issue bonds not to exceed 5% of the valuation of taxable property. As this indebtedness is incurred the Trustees provide for a direct annual tax sufficient to pay the interest and to discharge the principal of the indebtedness within at least twenty years. The Board of Trustees may also levy and collect annual taxes not to exceed 0.5% of the value of the taxable property. In addition, the Board may levy and collect a like sum, provided this additional annual tax has been authorized by the voters.

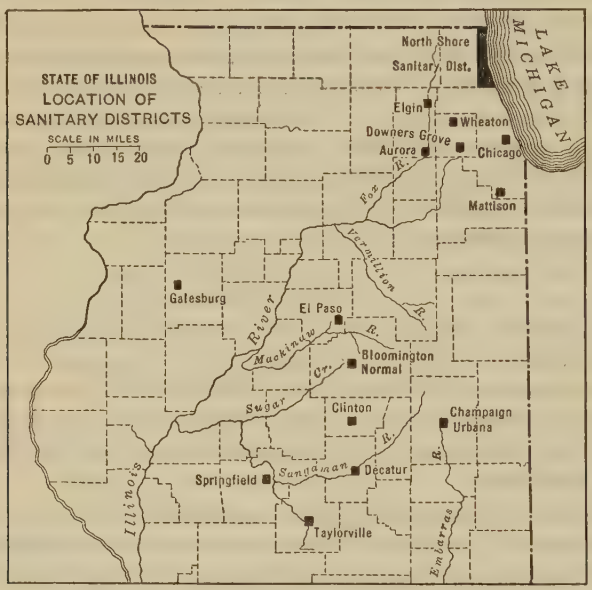

FIG. 1.

(*f*) The Trustees are given power to prevent the pollution of any waters from which a water supply may be obtained by any community in the District within a radius of fifteen miles from the intake of the water supply.

(*g*) The 1923 Amendment to this Act gives the Trustees authority to pay for the construction of sewers, drains, laterals, and appurtenances by special assessment in general accordance with the Act of 1897 concerning local improvements.

Up to 1924 bonds had been voted for the construction of sewage treatment projects in nine of the thirteen districts. Up to 1926 the total bonds so authorized amounted to $4 504 000. In addition, some money had been accumulated for construction purposes from annual taxes.

TABLE 1.—SANITARY DISTRICTS OF ILLINOIS. FINANCIAL DATA.

District.	Year of data.	Approximate population.	Assessed valuation.	Bond Limit.		Bonds authorized.	Approximate Construction Cost.		Construction cost per capita.
				Total.	Per capita.		Sewers and appurtenances.	Sewage treatment works.	
North Shore	1925	58 000	$28 952 020	$1 447 601	$25.00	None			
Decatur	1923	48 000	17 791 937	889 597	18.50	$860 000	$752 848*	$569 904*	$27.60
Bloomington-Normal	1925	36 000	16 528 549	826 427	23.00	700 000	144 178*	596 310*	20.60
El Paso	1920	1 638				43 000			
Urbana-Champaign	1926	40 000	13 000 000	650 000	16.25	500 000	89 500*	397 884*	12.20
Downers Grove	1921	3 540	1 500 000	75 000	21.20	75 000	15 000*	50 000*	18.35
Elgin	1925	30 000	14 475 128	723 757	24.10	700 000	268 500*	435 210*	23.40
Taylorville	1926	10 000	2 523 504	126 175	12.62	126 000	61 445*	30 940*	9.24
Springfield	1924	60 000	32 656 746	1 632 838	27.21	1 500 000	1 603 000*	525 000†	35.50
Galesburg	1923	25 000	12 000 000	600 000	24.00	None	450 000†	100 000†	22.00
Wheaton	1926	7 500‡	3 018 164	150 000	20.00‡	120 000	106 900	129 500	45.00‡
Clinton	1926	7 500	2 311 200	130 000	17.30	None			
Aurora	1926	41 000	20 000 000	1 000 000	24.40	None			

* Contracts.

† Estimated.

‡ Design population estimated for 1930.

TABLE 2.—Sanitary Districts of Illinois. General Data.

District.	Year formed.	Area included, in square miles.	Population.			Population per Acre.		
			1920 Census.	Treatment plant, capacity.	Intercepting sewer, capacity.	1920 Census.	Treatment plant, capacity.	Intercepting sewer, capacity.
North Shore	1914	25.0	36 000	.....	.....	2.3	....	....
Decatur	1917	33.0	43 818	60 000	120 000	2.1	2.8	5.7
Bloomington-Normal	1919	8.3	33 868	50 000	75 000	6.4	9.4	14.1
El Paso	1919	2.3	1 638	.....	.....	1.1	....	....
Urbana-Champaign	1921	8.5	35 000	45 000	70 000	6.4	8.3	12.8
Downers Grove	1921	5.6	3 543	5 000	25 000	1.0	1.4	7.0
Elgin	1922	8.3	28 260	37 500	75 000	5.3	7.0	14.0
Taylorville	1923	3.5	7 000	4 500	.....	3.1	....	....
Springfield	1924	36.2	63 000	90 000	193 000	2.7	3.9	8.3
Galesburg	1924	17.2	23 834	30 000	70 000	2.2	2.7	6.4
Wheaton	1925	15.5	4 137	10 000	.....	.4	1.0	....
Clinton	1925	1.6	5 898	.....	.....	5.9	....	....
Aurora	1925	11.6	36 397	.....	.....	4.9	....	....
Matteson	1925	....	.....	.....	.....	....	....	....
Hinsdale	1926	....	.....	.....	.....	....	....	....

The data shown in Table 1 indicate that the amount of bonds allowed Illinois Sanitary Districts is in general not sufficient to meet the cost of intercepting sewers and sewage treatment works. At Decatur, where the cost of sewage disposal works is increased by the industrial sewages, the amount of money available from bonds has been but little more than one-half the total necessary expense. The difference has been secured by accumulating the surplus from annual taxes.

General statistics of a number of the Illinois Sanitary Districts are shown in Table 2. The general location of the sewage treatment projects of the North Shore Sanitary District is shown in Fig. 2.

TABLE 3.—Springfield, Illinois, Measured Sewage Flow. Summary Table.

Sewer.	Dates.	Sewage Flow Rates.				
		Average per capita per day.	Maximum Hour.		Minimum Hour.	
			Per capita per day.	Percentage of average.	Per capita per day.	Percentage of average.
Ridgeley*	October 9-11	82	114	139	62	76
Amos Branch†	October 12-14	42.5	127	300	20	47
Town Branch§	October 20-23	155	217	140	112	72
Cook Street (wet weather)‡	October 23-25	148	176	119	125	85
Cook Street (dry weather)*	November 18-19	89	103	116	77	87
Oak Knowles§	October 26-28	149	153	103	140	94

* Dry-weather flow.

† Some rain during gaugings—dry flow.

‡ Wet-weather flow.

§ Sewers located in natural valley. Flows probably affected by sub-surface drainage from large area.

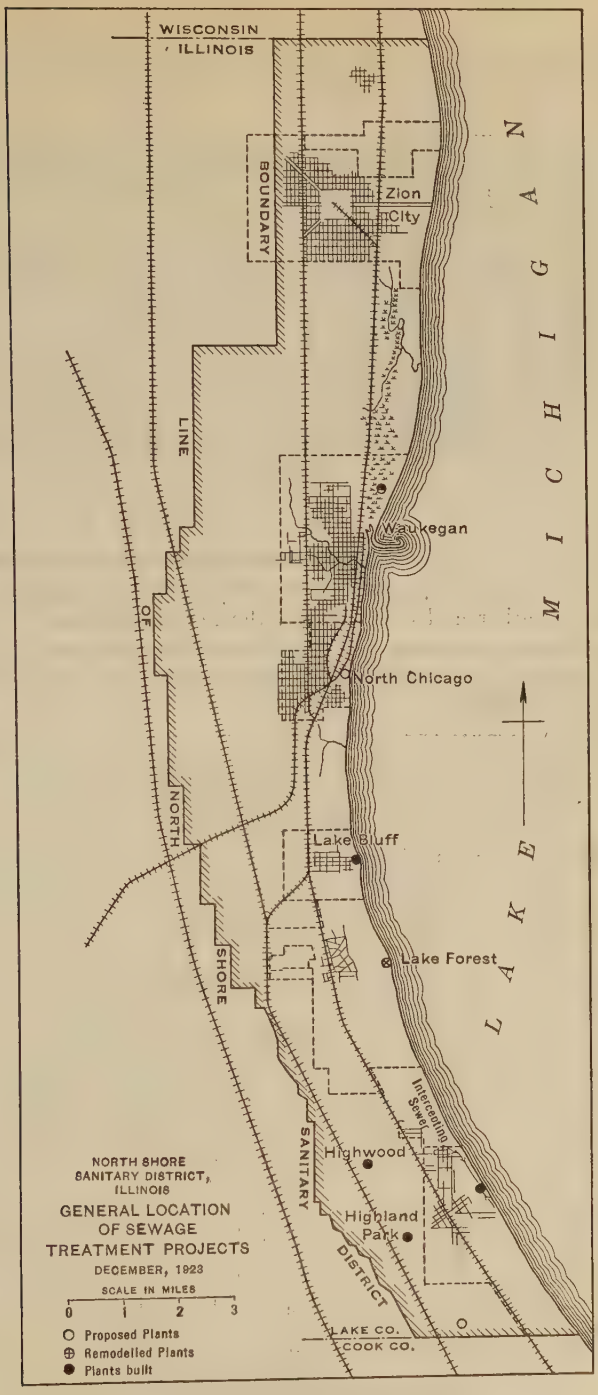
WISCONSIN
ILLINOIS
BOUNDARY
LINE
OF
NORTH
SHORE
SANITARY
DISTRICT
Zion
City
Waukegan
North Chicago
Lake Bluff
Lake Forest
Highwood
Highland
Park
Intercepting
Sewer
LAKE
MICHIGAN
LAKE CO.
COOK CO.
NORTH SHORE
SANITARY DISTRICT,
ILLINOIS
GENERAL LOCATION
OF SEWAGE
TREATMENT PROJECTS
DECEMBER, 1923
SCALE IN MILES
0 1 2 3
Proposed Plants
Remodelled Plants
Plants built

FIG. 2.

Sewage Quantities

In many of the districts, weirs have been built at the end of one or more main sewers for measuring the sewage flow. Table 3 gives the results of gaug-

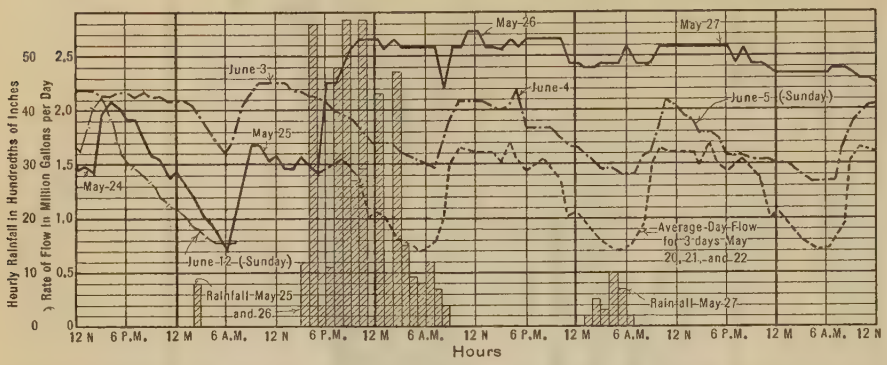
Hourly Rainfall in Hundredths of Inches
Rate of Flow in Million Gallons per Day
May-26
May-27
June-3
June-4
June-5 (Sunday)
May-25
May-24
Average-Day-Flow for 3 days May 20, 21, and 22
June-12 (Sunday)
Rainfall-May-25 and 26
Rainfall-May-27
12 N 6 P.M. 12 M 6 A.M. 12 N 6 P.M. 12 M 6 A.M. 12 N 6 P.M. 12 M 6 A.M. 12 N 6 P.M. 12 M 6 A.M. 12 N
Hours

Fig. 3.—Effect of Rain on Sewage Flow, Champaign Outlet Sewer, During 1921.

ings at Springfield in 1923. The Cook Street Sewer has a drainage area of 916 acres and an estimated population of 8 710 (1923). Gaugings were made following rainy weather and dry weather, the average sewage flow increasing from 89 to 148 gal. per capita per 24 hours during the wet season. Wet-season flows appear to be considerably higher than dry-season flows in the Illinois Sanitary Districts.

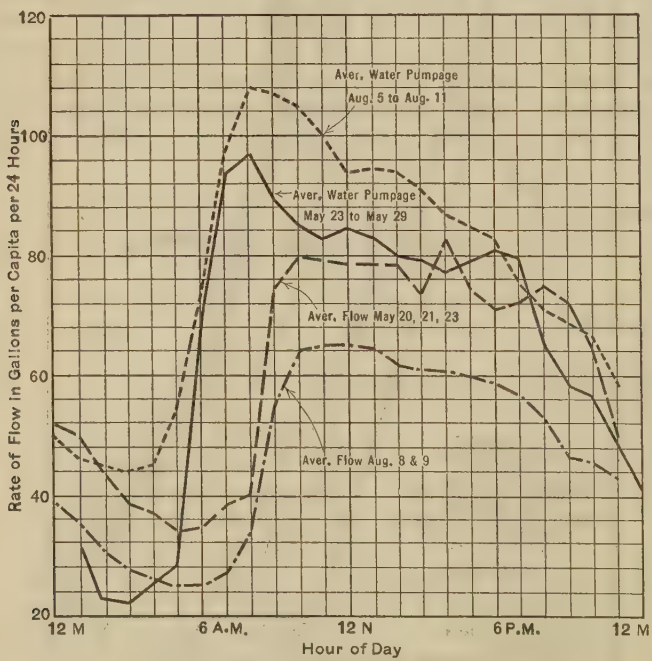
Rate of Flow in Gallons per Capita per 24 Hours
Aver. Water Pumpage Aug. 5 to Aug. 11
Aver. Water Pumpage May 23 to May 29
Aver. Flow May 20, 21, 23
Aver. Flow Aug. 8 & 9
12 M 6 A.M. 12 N 6 P.M. 12 M
Hour of Day

Fig. 4.—Hourly Rates of Sewage Flow, Champaign Outlet Sewer, During 1921.

At Urbana-Champaign, the State Water Survey maintained a weir at the end of the Champaign Sewer for almost a year. Fig. 3 shows the rates of sewage flow following rains. The lower broken line indicates typical dry-season hourly rates of flow. The upper lines show the rates following (not during) rains. It took from May 27 to June 9 (13 days) for the sewage flow to reach normal dry-weather rates. This system was built as separate sewers, but appears to receive some surface sewage. Typical wet and dry-season hourly rates of sewage flow and also of the water pumpage at Champaign are shown in Fig. 4.

Fig. 5 shows hourly rates of sewage flow as gauged at Elgin. All the per capita rates are quite low except the West Chicago Street Sewer, which receives the sewage from a malted milk factory, totaling about 650 000 gal. per day.

All these gaugings have been studied and average rates of sewage flow determined. These rates, with data on the design of intercepting sewers, are given in Table 4. The Urbana-Champaign sewers are the only ones built as a separate system. In Elgin about one-half the city is served with separate sewers. The other districts have combined systems.

TABLE 4.—Illinois Sanitary Districts. Design Data for Intercepting Sewers.

Sanitary district.	Population per Acre.		Drainage area of stream receiving overflow, in square miles.	Gallons per Capita per 24 Hours.			Type of sewer system.
	Present.	Basis of design.		Average water consumption.	Average sewage flow.	Capacity of intercepting sewer.	
Decatur	2.1	5.7	862	105*	125	375	Combined
Urbana-Champaign	6.4	12.8	76	40	75	136	Separate
Elgin	5.3	13.0	1 520	40	100	261–620†	Half Combined Half Separate
Springfield:							
West Side	8.7 (‡)	17.0‡	265	71	125	425	Combined
East Side	8.7 (‡)	17.0‡	110	71	125	2 125–300§	Combined
Galesburg	4.1	14.0	10	66	78	500	Combined

* Metered consumption, not the total pumpage.

† 620 gal. per capita per day in 1926; 261 gal. per capita per day in 1964.

‡ On basis of sewered areas.

§ 2 125 gal. per capita per day carried to future storm-water treatment site; 300 gal. per capita per day from there to main treatment plant.

Sewage Characteristics

During the gaugings of the sewers, composite samples of the sewage were taken. Table 5 shows the results of analyses at Springfield. This is one of the weakest of Illinois sewages. During the operation of a testing station by the State Water Survey on the Champaign sewage in 1921, gaugings and chemical analyses were made. Fig. 6 shows the relation between rainfall, sewage flow, and sewage characteristics over an 8-month period. During periods of low flow, the oxygen consumed and the total organic nitrogen increased.

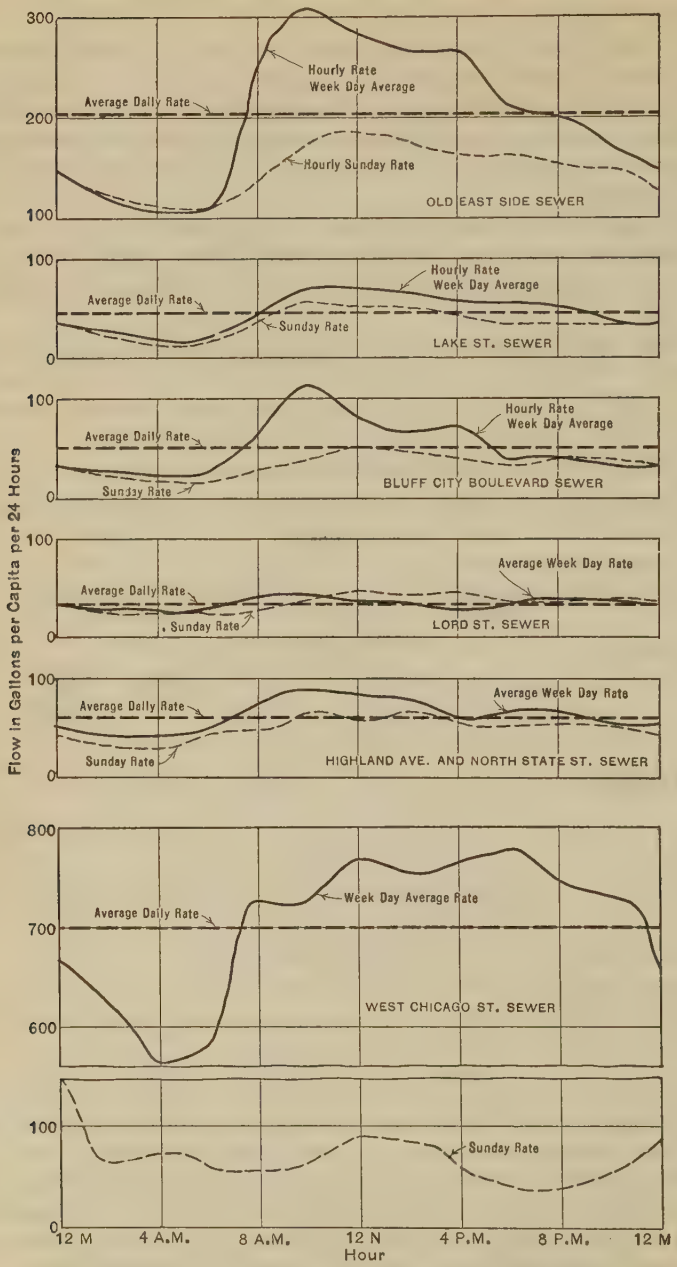

FIG. 5.—DRY-WEATHER SEWER GAUGINGS AT ELGIN, ILL.

Typical results of sewage analyses are shown in Table 6. At Decatur the domestic sewage amounts to about 5 500 000 gal. per day to which is added about 4 500 000 gal. per day of strong starch-works sewage. The analyses typical of domestic sewage were taken when the starch works was shut down.

TABLE 5.—Illinois Sanitary Districts. Summary of Chemical Analyses for Various Sewages at Springfield.

(Results given in parts per million.)

Sanitary district.	Date.	Average flow, in gallons per capita per 24 hours.	Total solids.	Suspended solids.	Nitrogen.		Oxygen consumed.	Alkalinity as $CaCO_3$.
					Total organic.	Free NH_3.		
Springfield, Ill.:								
Town Branch	Oct. 10–27, 1923*	154	1 050	46	9.3	12.0	46.0	350
	Nov. 9–10, 1923†	....	1 012	146	....	....	82.0	....
Amos Branch	Oct. 10–27, 1923*	42	906	92	24.0	34.0	64.2	412
Ridgely	Oct. 10–27, 1923*	82	924	4	4.3	5.6	27.2	330
	Nov. 9–10, 1923†	....	1 158	30	....	....	30.6	....
Cook Street	Oct. 10–27, 1925*	147	1 398	72	5.2	2.4	28.8	340
	Nov. 9–10, 1925†	88	1 400	30	....	....	29.2	....
Governor Street	Oct. 10–27, 1925*	144	.....	36	6.0	6.4	25.7	317
	Nov. 9–10, 1925†	....	656	40	....	....	18.0	....
Typical (Combined)	.....	125	1 000	72	8.1	9.6	48.0	344

* Each analysis represents a 24-hour composite sample taken hourly.

† Each analysis represents a 24-hour composite sample of samples taken at 30-min. intervals.

Intercepting Sewers

With most of the sewers combined, intercepting sewers have been proportioned to take the normal flow and the first run-off of rains. The design has been based on a per capita sewage flow from the future estimated population in accordance with the data shown in Table 4.

TABLE 6.—Illinois Sanitary Districts. Average Analyses of Sewage in Several Illinois Sanitary Districts.

Sanitary district.	Suspended matter.	Nitrogen.		Oxygen consumed.	Alkalinity.	Average sewage flow, in gallons per capita per 24 hours.
		Total organic.	Free ammonia.			
Decatur: Domestic, 1924...	240	9.6	18.4	61	368	125
Decatur: Combined, 1924*.	291	77	17.0	380	...	200
Springfield	72	8.1	9.6	48	344	125
Urbana-Champaign	...	12.3	16.4	59	420	75
Elgin	296	29.0	23.0	149	532	100

* Varies with operations at starch works.

In general, intercepting sewers for combined systems and with overflows into relatively small streams have been given capacities for 400 gal. or more per capita every 24 hours. At Springfield, special conditions required a larger

capacity. Four combined sewers discharged 10 miles or more by water above a proposed water-works impounding dam. The cost of diverting the entire flow to below the dam was very great (more than $2 250 000).

It was decided to intercept 2 125 gal. per capita per 24 hours from these sewers and to bring this quantity to one point for treatment. The first flow up to 300 gal. per capita per 24 hours will be pumped to a point below the proposed dam and there passed through complete treatment works. The overflow will be chlorinated near the pumping station. In some other cases additional capacity has been allowed for special industrial sewages.

Treatment Plant Loadings

The capacities of the major parts of a number of the sewage treatment plants are shown in Table 7. The capacities were fixed in accordance with the rates of sewage flow, the characteristics of the sewage, the estimated future population, the character of the site, and similar factors. The sewage treatment works of the North Shore District have the largest unit capacities. In that District the rates of sewage flow are extremely variable, the sewage is quite fresh, and the plant sites are in the midst of high-class residences. It was considered wise to allow quite liberal capacities.

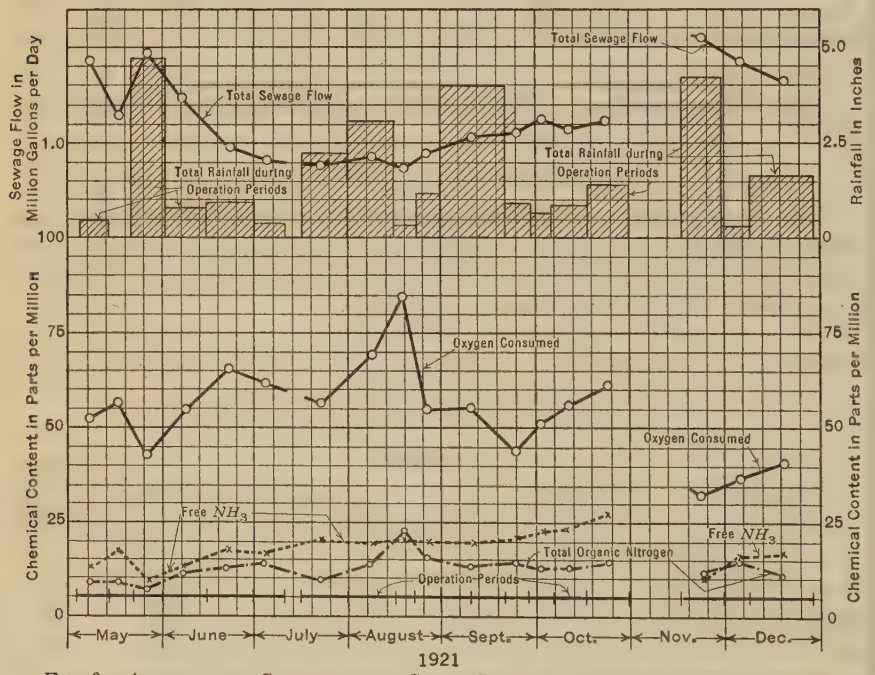

Fig. 6.—Amount and Character of Crude Sewage, Champaign Outlet Sewer.

At Decatur, the sewage flow has increased to about 10 000 000 gal. per day, or about 25% more than the rated capacity of the plant. At this rate of flow, the displacement period in the Imhoff tanks is about 1.15 hours. However, during more than a year of operation, these tanks have reduced the suspended

TABLE 7.—ILLINOIS SANITARY DISTRICTS. SEWAGE TREATMENT PLANTS, ELEMENTS OF IMHOFF TANKS, AND SPRINKLING FILTERS.

Elements of design.	Downers Grove.	Decatur.	Urbana-Champaign.	North Shore Sanitary District.				Elgin, from design plans.
				Deerfield.	Highwood.	Park Avenue.	Lake Bluff.	
Year built	1921	1922-24	1923-24	1923	1923	1923	1923	1925-26
Population:								
Actual	3 550	43 000	33 000	640	1 500	2 800	700	28 500
Basis of design	5 000	60 000	45 000	1 000	2 000	3 600	1 460	37 500
Sewage Flows, in 1 000 000 Gal. per Day:	.....	7.96	1.96	0.20	0.40	0.72	0.28	3.31
Design basis, gallons per capita per day	.....	132.5	43.5	200	200	200	200	88
Settling Capacity, in Gallons:	45 050	476 000	284 400	33 525	68 850	88 740	46 820	411 200
Hours (actual)	.....	1.41	3.04	4.02	4.12	2.96	4.0	2.99
Gallons per capita	9.01	7.92	5.52	33.5	34.4	24.6	33.4	11.0
Sludge Capacity, in Cubic Feet:	10 000	181 900	99 720	4 337	7 084	12 230	4 688	80 400
Per capita	2.0	3.03	2.22	4.34	3.54	3.4	3.35	2.15
Gas Vent Area, in Square Feet:	132	4 980	2 728	237	316.4	559	300	1 980
Percentage of tank area	17.2	31.1	31.7	29.7	26.4	22.3	29.7	24.2
Maximum water depth, in feet	31.6	27.06	28.25	24.75	28.0	22.0	25.5	29.5
Sludge Bed Area:								
Total square feet	2 500	40 000	30 000	840	1 612	3 892	1 585	24 750
Per capita	0.50	0.67	0.67	0.84	0.81	1.08	1.13	0.67
Sprinkling Filter:								
Acres	0.268	3.04	1.60	0.069	0.128	None	None	1.54
Depth, in feet	7.25	6.0	10.0	6.0	6.0	.....	.....	8.0
Population Loading:								
Per acre	18 650	19 700	28 150	14 600	15 600	.....	.....	24 400
Per acre-foot	2 572	3 280	2 815	2 770	2 600	.....	.....	3 050

matter by about 75% by weight, delivering an effluent with an average of 70 parts per million of suspended matter. The rate of sewage flow through the plant at Decatur is relatively uniform throughout the 24 hours, ranging from 8 000 000 to 11 000 000 gal. per day. The strength of the sewage adds a further burden on the sprinkling filter which is to be handled through the construction in 1927 of a pre-aeration plant.

TABLE 8.—Springfield, Illinois, Estimated Unit Construction Cost of Sewers.

Diameter of sewer.	Estimated Cost per Linear Foot.					
		Masonry and Excavation.				
	Masonry (open cut).	Open Cut.				Tunnel.
		0–6–ft. cut.	6–12–ft. cut.	12–18–ft. cut.	18–24–ft. cut.	
12 in.	.....	$ 0.95	$ 1.25	$ 2.00	$ 3.00	.....
15 in.	.....	1.30	1.65	2.60	3.20	.....
18 in.	.....	1.65	2.05	3.00	4.40	.....
21 in.	.....	2.30	2.75	3.90	5.70	.....
24 in.	.....	2.90	3.50	4.70	6.80	.....
27 in.	.....	3.70	4.45	5.70	8.50	.....
30 in.	$ 4.20	5.00	6.25	7.50	9.50	$ 27.00
33 in.	4.30	5.25	6.50	8.00	10.00	28.00
36 in.	4.40	5.50	6.75	8.50	10.50	29.00
39 in.	4.50	6.00	7.25	9.25	11.50	30.00
42 in.	5.00	6.50	7.90	10.00	12.50	31.00
45 in.	5.50	7.00	8.50	10.75	13.50	32.00
48 in.	6.00	7.50	9.25	11.50	14.50	33.00
51 in.	6.30	8.00	9.75	12.25	15.25	33.75
54 in.	6.60	8.50	10.25	13.00	16.00	34.75
57 in.	7.00	9.00	11.00	13.75	17.00	35.75
60 in.	7.70	9.50	11.50	14.50	18.00	36.75
66 in.	8.50	10.50	13.00	16.00	20.00	38.75
72 in.	9.30	11.50	14.00	17.50	21.75	40.50
78 in.	15.00	17.25	19.50	23.25	27.25	42.50
84 in.	17.00	19.00	21.75	25.50	29.50	45.00
90 in.	19.00	21.25	24.00	27.75	32.25	47.25
96 in.	21.00	23.50	26.50	30.00	35.00	49.50
9 ft.	27.00	29.00	32.00	36.00	41.50	55.00
10 ft.	32.00	35.00	38.00	42.50	49.00	62.00
11 ft.	38.00	41.00	45.00	50.00	56.50	69.50
12 ft.	44.00	47.50	52.00	57.50	64.50	78.00
13 ft.	50.00	54.50	59.00	65.00	72.50	86.50
14 ft.	58.00	62.00	66.00	73.00	81.00	95.00
15 ft.	64.00	68.00	73.50	80.00	89.00	104.00
16 ft.	70.00	75.00	80.00	87.50	96.50	112.00
17 ft.	77.50	82.50	88.50	96.00	105.00	121.00
18 ft.	86.00	91.00	97.50	105.50	115.50	129.00
19 ft.	95.00	100.00	106.50	115.00	125.50	137.50
20 ft.	100.00	105.50	112.50	122.00	133.00	146.00

Construction Costs

Many miles of intercepting sewers have been built under a wide variety of construction conditions ranging from tunnel work in rock or water-bearing

TABLE 9.—ILLINOIS SANITARY DISTRICTS. TYPICAL UNIT COSTS OF SEWAGE TREATMENT WORKS.

District.	Year.	Basis Design.		Cost per Million Gallons per Day.					Cost per Capita.				
		Population.	Average flow, in million gallons per day.	A.	B.	C.	D.	E.	A.	B.	C.	D.	E.
Decatur	1923	60 000	7.96	.....	$20 400	$41 400	$9 600	$71 400	.....	$2.70	$5.50	$1.26	$9.46
Urbana-Champaign	1923	45 000	1.96	$21 000	66 000	126 000	5 260	218 000	$0.92	2.86	5.50	0.23	9.51
Elgin	1925	37 500	3.31	25 200	33 100	62 600	17 100	128 000	2.22	2.92	5.52	1.51	12.17
Downers Grove	1922	5 000	0.50	12 000	25 000	58 000	5 000	100 000	1.20	2.50	5.80	0.50	10.00
Wheaton	1925	7 500	1.00	.....	.....	.....	.....	129 500	.....	.....	.....	1	17.20

NOTE.—(A), Pumping stations; (B). Imhoff tanks, sludge beds, and appurtenances; (C), sprinkling filters, secondary tanks, and appurtenances; (D), grit basins, laboratory buildings, and miscellaneous items; and (E), total.

gravel to open-cut work along the Fox River at Elgin. Space does not permit a complete statement regarding unit costs for sewer construction, but recent bids received at Springfield for intercepting sewer work aggregated about $1 500 000, and have been quite close to the estimates in the preliminary report. Table 8 shows these estimated costs of sewers of different sizes for different depths of cut. To these unit prices additional cost items were added for manholes, railroad crossings, rock excavation, etc. Later bids have been lower.

Table 9 shows unit costs of construction for several completed projects. These costs should be scrutinized in conjunction with the treatment plant loadings of Table 7. The unit figures are computed from the bids. The construction work in each case was good and the contractors' prices were fair. Extras at the finished plants at Decatur and Urbana amounted to about 0.3 per cent. Costs for complete treatment works, including Imhoff tanks, sprinkling filters, and appurtenances (but without pumping stations), are indicated as about $10 per capita.

Sprinkling Filter Stones

Illinois and Indiana limestones have been used for the sprinkling filters. The selection was based on cost and on a careful study of experience with similar stone in operating filters. A record of the chemical analyses of several of these filter stones is given in Table 10.

TABLE 10.—Illinois Sanitary Districts. Analyses of Sprinkling Filter Stones.

Item.	Decatur.*	Deerfield Avenue Plant, North Side Sanitary District.†	Urbana-Champaign.‡	Elgin.§
Calcium carbonate	51.93%	54.24%	96.49%	46.71%
Magnesium carbonate	37.81%	42.78%	1.47%	38.95%
Silica	4.08%	0.68%	1.60%	11.78%
Alumina	5.16%	2.23%	.....	1.82%
Iron	0.48%	Trace	.....	0.45%
Iron and alumina	.....	.....	0.88%	.....
Sulfur	.....	.....	.....	0.17%
Lime (calcium sulphate)	.....	.....	.....	0.06%
Totals	99.41%	99.93%	99.94%	99.94%

* Material from Lehigh Stone Company, Kankakee, Ill., analysis given in average of two analyses.

† Average of analyses of two samples.

‡ Analysis of stone from Greencastle, Ind., quarry of Ohio-Indiana Stone Company. Part of the filter stone used was obtained from the Lehigh Stone Company, Kankakee, Ill.

§ Average of analyses of three samples.

The stone was delivered to the filters by truck or by industrial car. After screening to size at the quarry, the material was screened again at the site of the work to remove small particles and stone dust. If the stone was wet so that too much dust adhered to the surface, it was washed. Approximate figures for the cost of filter stone in place are given in Table 11.

TABLE 11.—Approximate Cost of Filter Stone in Place.

District.	Material.	Cost per Cubic Yard.	
		Placing.	Total.
Decatur	.....	.....	$4.06
Urbana-Champaign	.....	.....	2.80
Elgin	$2.40	$1.10	3.50
Downers Grove	3.00	0.65	3.65

The bulk of the stone was between 1 in. and 2 in. in size. Larger stone up to 4¾ in. was used for the bottom 6 in. of the filters.

Treatment Plant Site Areas and Locations

Some data on the area and location of treatment plant sites may be of interest, as shown in Table 12. As available funds have permitted, work has been undertaken to improve the general appearance of the grounds about the treatment works through planting, roadways, ornamental lights, etc. As already noted, the treatment plants of the North Shore District are, in some instances, within 350 ft. of first-class residences. The others are not so critically located.

TABLE 12.—Illinois Sanitary Districts. Area and Location of Treatment Plant Sites.

District.	Distance from approximate center of city, in miles.	Area of plant site, in acres.	Direction of plant site from center of city.
Decatur	2.3	57	W.S.W.
Urbana-Champaign	1.4	51	E.N.E.
Elgin	1.3	39	S.
Downers Grove	1.5	7.5	W.
Springfield	3.0	....	N.
Wheaton	2.2	8.4	S.W.

Operating Results

A study of operating results is beyond the scope of this paper. The plants at Decatur and Urbana-Champaign have fully equipped laboratories and daily routine chemical analyses are made. In the large districts, the best of operation has been secured. In fact, the disposition of the District Trustees, to place plant operation in the hands of skilled and competent operators, constitutes one of the chief merits of the Illinois Sanitary District Act. This is reflected, not only in the character of plant superintendents, but also in the Association of Sanitary District Trustees, through which an earnest effort is made to further improve the results of the work done by the various districts. An annual meeting is held for the discussion of financial, construction, and

operation problems by the Trustees with their Superintendents and Engineers. Thus, successful progress and method in one District is made available to the Trustees of the other Districts.

The annual budget for the operation of the sewage treatment works at Decatur is itemized as follows:

1925-26 BUDGET

Payroll:	
Plant foreman and three men	$6 000
Extra summer help, three men	3 000
Supervision	4 000
Plant equipment and maintenance	2 000
Electric Power:	
Main Street Pump Station	1 500
Sewage Treatment Plant	1 500
Office supplies and printing	125
Laboratory equipment	250
Automobile	800
Landscaping	1 000
Gasoline and oil	300
Recording gauge	150
Contingencies	1 555
Total	$22 180

Table 13 gives typical monthly chemical analyses for two of the plants.

TABLE 13.—ILLINOIS SANITARY DISTRICTS. GENERAL ANALYTICAL DATA, SEWAGE TREATMENT PLANT OPERATION.

	PARTS PER MILLION.			
	Urbana-Champaign.		Decatur.	
	Raw sewage.	Final effluent.	Raw sewage.	Final effluent.
Suspended solids	130	.....	260	40
Oxygen consumed	.....	36.6	200	35
Ammonia nitrogen	12	13.5	32	.....
Organic nitrogen	10	6.4		
Nitrate nitrogen	1.26	9.4	.....	10-12
Nitrite nitrogen	1.60	4.56		
Biological oxygen demand	.....	.....	500-700	30-40

ACKNOWLEDGMENTS

Much of the information contained in this paper was furnished by the trustees and officers of the sanitary districts. Helpful assistance was given by principals and assistants in the writer's office, especially Messrs. R. S. Rankin and Paul E. Langdon, to whom grateful acknowledgment is made.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1635

Discussion*

THE DESIGN OF A MULTIPLE-ARCH SYSTEM AND PERMISSIBLE SIMPLIFICATIONS

By A. C. Janni, M. Am. Soc. C. E.

A. C. Janni,† M. Am. Soc. C. E. (by letter).—The writer at once wishes to call attention to a statement which, to do full justice to all, should have been incorporated in the original paper.‡ The statement was made at the time of presentation,§ as follows:

"The method followed [in the paper] is that of Professor Guidi of the Polytechnique of Turin. This material I have taken from his work of four or five volumes.‖ Incidentally, I would say that it is too bad that this work has not as yet been translated into English; it would be a great help to the Engineering Profession".

In view of certain remarks made by discussors, concerning some points of this paper, the writer has found it expedient to elucidate further on these points by extending his research so as to make more clear the legitimacy of this simplification.

Assume that Arch A_2 is the center arch of a five-arch system, A_0, A_1, A_2, A_3, A_4. For the sake of brevity assume that these five arches and their four piers, P_0, P_1, P_2, P_3, are of the same size as those shown in the case of a three-arch system.

Determination of the Ellipses of Elasticity of the Various Combinations of the Parts of This System

The ellipse of elasticity, G_{A_0}, of Arch A_0, is, of course, already known, also the ellipse of elasticity, G_{P_0}, of Pier P_0, as well as the ellipse of elasticity of the joint, $G_{A_0P_0}$, between Arch A_0 and Pier P_0.

* Published in May, 1927, *Proceedings*. Closing discussion of the paper by A. C. Janni, M. Am. Soc. C. E., continued from *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1244.

† Cons. Engr., New York, N. Y.

‡ *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1142.

§ At the meeting of the Society, September 3, 1924.

‖ "Lezioni sulla Scienza delle Costruzioni," by Prof. Camillo Guidi, Torino, Italy.

The determination of the ellipse, $G_{A_0P_0A_1}$ (combination of the two ellipses, $G_{A_0P_0}$ and G_{A_1}) is conducted as follows:

Let, I_x = moment of inertia with respect to the horizontal diameter, xx.
I_y = moment of inertia with respect to the vertical diameter, yy.
I_{xy} = product of inertia with respect to the diameters, xx and yy.
α = angle of inclination of the major axis to the horizontal.
I_a = moment of inertia with respect to the minor axis.
I_b = moment of inertia with respect to the major axis.
ρ_a = length of major axis.
ρ_b = length of minor axis.

Then,

$$I_x = 373.54 \times (1.30)^2 + 15.55 \times (6.17)^2 + 373.54 \times (0.05)^2 + 15.55 \times (1.22)^2 = 1247.05$$

$$I_y = 373.54 \times (12.90)^2 + 373.54 \times (1.02)^2 + 15.55 \times (1.09)^2 + 15.55 \times (24.65)^2 = 72022.01$$

$$I_{xy} = 1.22 \times 24.65 \times 15.55 + 1.02 \times 0.05 \times 373.54 = 491.67$$

$$\tan 2\alpha = -\frac{2 \times I_{xy}}{I_x - I_y} = 0.01389\,; \quad 2\alpha = 0^\circ\ 48'$$

$$I_a = \frac{1247.05 + 72022.01}{0.2} + \frac{1}{2}\sqrt{(70774.96)^2 + 4 \times (491.67)^2} = 72\ 025$$

$$I_b = \frac{1247.05 + 72022.01}{0.2} - \frac{1}{2}\sqrt{(70774.96)^2 + 4 \times (491.67)^2} = 1\ 243$$

$$\rho_a = \sqrt{\frac{72025}{389.09}} = 13.61 \text{ ft.}$$

$$\rho_b = \sqrt{\frac{1243}{389.09}} = 1.78 \text{ ft.}$$

Fig. 64 shows the graphical method used for the same determination (see Appendix, Paragraphs 8 and 9*). Naturally, the results are the same for both methods. In Fig. 64 is shown the relative position of the ellipse, $G_{A_0P_0A_1}$, with respect to the corresponding ellipse, G_{A_1} and $G_{A_0P_0}$.

It is to be observed that, unless the two elastic centers, $G_{A_0P_0}$ and G_{A_1}, are on the same horizontal line, the axes of the resulting ellipse, $G_{A_0P_0A_1}$, cannot be one horizontal and the other vertical; but they will be inclined to these directions by a certain angle, which, in this case, is 0° 24′. A glance at the expression for the value, d_1,† will show that this exceptional position of the two centers can never be verified.

In this case, however, as the angle made by the major axis of the resulting ellipse with the horizontal is very small, it may be assumed to be zero without appreciable error in the results, and, in so doing, all complications in the next determination of the ellipse of the joint between $G_{A_0P_0A_1}$ and G_{P_1} resulting from this inclination of the axes may be avoided.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), pp. 1164–1165.
† *Loc. cit.*, p. 1152.

The determination of this last ellipse, $G_{A_0P_0A_1P_1}$, is made in the same way as the determination of the similar ellipse of the preceding joint. The result is shown in Fig. 65, the values referred to the ellipse being:

$$d_1 = 16.80 \text{ ft}; \ \rho = 7.65 \text{ ft}; \ G_{A_0P_0A_1P_1} = 16.20$$
$$d_1' = 0.006 \text{ ft}; \ \rho' = 1.07 \text{ ft}.$$

Analysis of Arch A_2 (Center Arch of a System of Five Arches)

Unyielding Foundations.—The analysis of Arch A_2 is carried out exactly as in the paper (in which A_2 was the central arch of a three-arch system), the only difference being that in the present instance, Ellipse $G_{A_0P_0A_1P_1}$ takes the place of Ellipse $G_{A_1P_1}$.

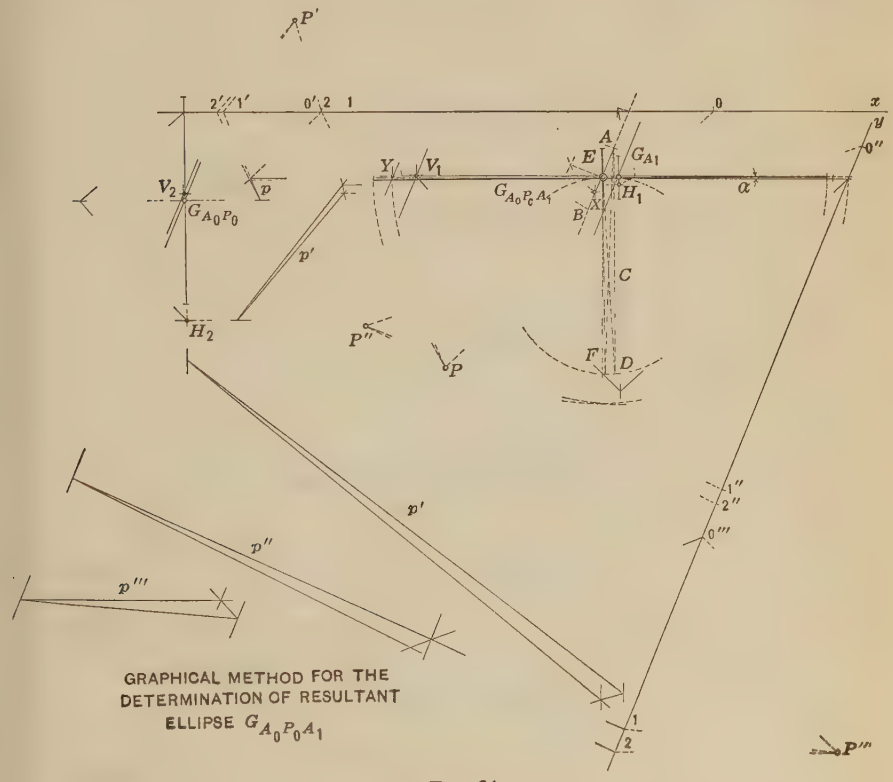

Fig. 64.

Incidentally, in the drawing of the five diagrams (Polygons p_1, p_2, p_3, p_4, and p_5), there is a simplification, which the writer would suggest, and which may be applied in case this method of design is used. A certain arrangement of computations was suggested in the paper.* Similar computations for the present Arch A_2 are as given in Table 12.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1146.

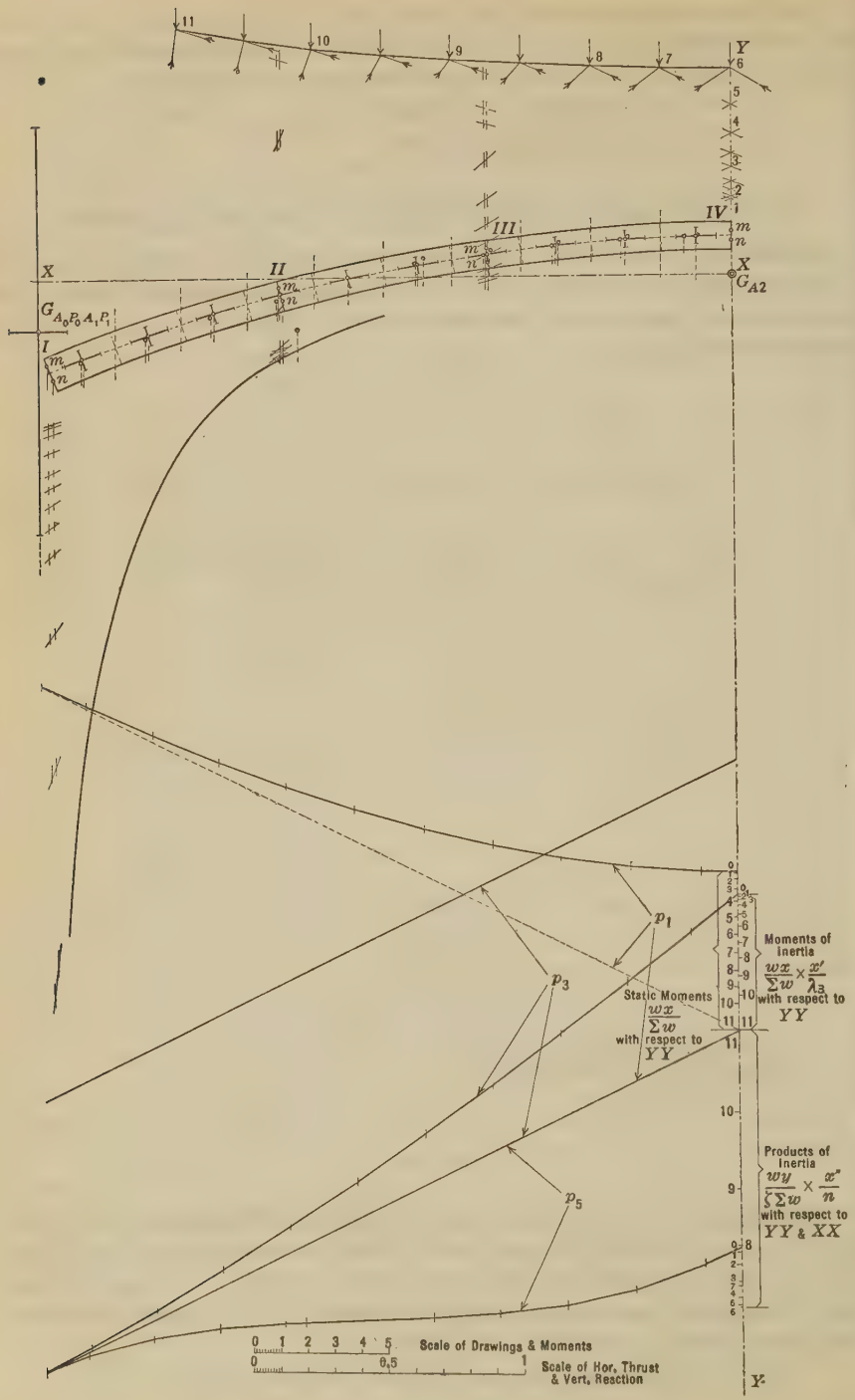

FIG. 65.—DESIGN OF A MULTIPLE-ARCH SYSTEM.

TABLE 12.—COMPUTATIONS OF ELASTIC QUANTITIES FOR CENTER ARCH A_2, OF FIVE-ARCH SYSTEM, UNYIELDING FOUNDATIONS.

Position of load.	w. (1)	x. (2)	y. (3)	x'. (4)	y'. (5)	x''. (6)	$\frac{w\,x}{\Sigma\,w}$. (7)	$\frac{w\,y}{\zeta\,\Sigma\,w}$. (8)	$\frac{w\,x}{\Sigma\,w}\times\frac{x'}{\lambda_8}$. (9)	$\frac{w\,y}{\zeta\,\Sigma\,w}\times\frac{y'}{v}$. (10)	$\frac{w\,y}{\zeta\,\Sigma\,w}\times\frac{x''}{n}$. (11)
1	2.55	0.64	0.71	0.85	0.73	0.63	0.04	0.31	0.003	0.04	0.08
2	2.41	1.94	0.66	2.03	0.70	1.93	0.11	0.27	0.022	0.03	0.23
3	2.28	3.23	0.56	3.29	0.60	3.20	0.18	0.22	0.059	0.02	0.32
4	2.15	4.51	0.41	4.56	0.48	4.44	0.23	0.15	0.104	0.01	0.30
5	2.03	5.79	0.21	5.84	0.31	5.66	0.29	0.07	0.169	0.004	0.18
6	1.93	7.06	—0.03	7.10	— 1.00	8.02	0.33	—0.01	0.234	0.001	—0.03
7	1.72	8.31	—0.33	8.31	— 0.43	8.40	0.35	—0.10	0.290	0.007	—0.38
8	1.38	9.57	—0.66	9.57	— 0.74	9.60	0.32	—0.16	0.306	0.02	—0.70
9	1.19	10.81	—1.06	10.81	— 1.11	10.83	0.31	—0.22	0.335	0.04	—1.09
10	1.03	12.03	—1.50	12.03	— 1.54	12.04	0.30	—0.26	0.360	0.07	—1.43
11	1.62	12.83	—0.94	12.86	—16.78	12.83	0.51	—0.26	0.655	0.86	—1.53
....	20.29 / 2	.....	.....	.....	.....	.....	.....	.....	2.537 / 2	1.091 / 2	.....
....	40.58	.....	.....	.....	.....	.....	.....	.....	$v = 5.074$ in.	$n = 2.182$ in.	.....

From Table 12, the following results are obtained,

$$\lambda_3 = 10.00 \text{ in.}; \zeta \Sigma w = \frac{40.58}{7} = 5.79$$

$$\rho = \sqrt{\frac{1}{7} \times 5.07 \times 2.18} = 1.19 \text{ in.}$$

$$\rho_1 = \sqrt{\frac{5.07 \times 10.00 \times 40.58}{40.58}} = 7.12 \text{ in.}$$

It is to be noted that, except for Columns (2), (3), (4), (5), and (6), which contain distances taken directly from Fig. 65, the remaining Columns (7), (8), (9), (10), and (11), are computations made by the slide-rule and represent: Column (7), the static moments (intercepts on $Y\,Y$) of the various elastic weights with respect to $Y\,Y$; Column (8), the static moments (intercepts on $X\,X$) of the same elastic weights with respect to $X\,X$; Column (9), the moments of inertia (intercepts on $Y\,Y$) of the same elastic weights with respect to $Y\,Y$; Column (10), the moments of inertia (intercepts on $X\,X$) of the same quantities with respect to $X\,X$; and, finally, Column (11), the products of inertia of the same quantities with respect to $X\,X$ and $Y\,Y$.

Thus it is clear that the drawing of Polygons p_2 and p_4 (Columns (8) and (10)), which was done for demonstration, may be omitted in practical cases in which usually the assumed loading is vertical. After all, Polygons p_2 and p_4 are drawn solely to obtain the quantities, $\frac{w\,y}{\zeta \Sigma w}$ and n, which are needed to draw Polygon p_5; these quantities can be had from Columns (8) and (10) as shown. The position of the elastic center of the arch can be determined by simple computations.

Furthermore, there are other simplifications. Polygons p_1, p_3, and p_5 (Columns (7), (9), and (11)), can be drawn without drawing first the relative force polygons. In fact, Column (7) gives the static moments of w with respect to $Y\,Y$. The corresponding lengths can be laid off on the axis, $Y\,Y$, as shown on Fig. 65 and the various sides of Polygon p_1 can then be drawn. Similarly, the two other polygons, p_3 and p_5, can be drawn by joining points with lines, rather than by constructing parallels to the radii of the force polygons, P_3 and P_5.

Fig. 65 shows this simplified and shortened method for obtaining the three diagrams, p_1, p_3, and p_5. These simplifications reduce the graphical work considerably.

The writer will not explain again the method followed for obtaining the influence and intersection lines, for it is exactly the same as shown in the paper; neither will he explain again the method for obtaining Table 13, which gives the moment for Sections I, II, III, and IV on the hypothesis that Arch A_2 is the center arch of a system of five arches. By following the same reasoning as that for a three-arch system, the forces acting on Arches A_1 and A_0 and on Piers P_1 and P_0, when Arch A_2 is loaded, have been determined (Tables 14 and 15). Fig. 66 shows the results obtained.

TABLE 13.—Computations for Moments in Arch A_2 (Center Span of Five-Span System), With Unyielding Foundations.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.108	− 2.25	− 1.80	−0.243	−0.194	−2.20	−1.80	−0.237	−0.194	−1.19	−0.80	−0.128	−0.086	+1.17	+1.52	+0.126	+0.164
2	0.320	− 2.55	− 2.07	−0.816	−0.664	−2.30	−1.86	−0.736	−0.595	−1.05	−0.70	−0.336	−0.224	+1.50	+1.85	+0.480	+0.592
3	0.547	− 3.20	− 2.72	−1.750	−1.490	−2.42	−2.00	−1.326	−1.094	−0.77	−0.40	−0.423	−0.218	+2.35	+2.70	+1.285	+1.476
4	0.729	− 4.00	− 3.50	−2.916	−2.551	−2.55	−2.07	−1.858	−1.512	−0.23	+0.14	−0.167	+0.102	+3.60	+3.95	+2.624	+2.879
5	0.783	− 4.55	− 4.05	−3.562	−3.171	−2.56	−2.06	−2.004	−1.612	+0.22	+0.65	+0.215	+0.508	+4.65	+5.00	+3.640	+3.915
6	0.798	− 5.20	− 4.70	−4.149	−3.750	−2.55	−2.05	−2.034	−1.635	+0.92	+1.30	+0.734	+1.037	+6.05	+6.40	+4.827	+5.107
7	0.783	− 6.05	− 5.50	−4.737	−4.306	−2.40	−1.92	−1.879	−1.503	+1.85	+2.25	+1.448	+1.761	.....	.....	.....	.....
8	0.729	− 7.10	− 6.55	−5.175	−4.774	−2.12	−1.62	−1.545	−1.180	+3.30	+3.70	+2.405	+2.697	.....	.....	.....	.....
	0.603	.....	.....	.....	.....	.....	.....	.....	.....	.....	+7.32	.....	+4.413	.....	.....	.....	.....
	0.601	.....	.....	.....	.....	.....	.....	.....	.....	+6.95	.....	+4.176	.....	.....	.....	.....	.....
9	0.547	−10.15	− 9.50	−5.552	−5.196	−0.57	−0.07	−0.311	−0.038	+6.55	+6.85	+3.582	+3.746	.....	.....	.....	.....
10	0.320	−15.35	−14.50	−4.912	−4.640	+5.45	+6.12	+1.744	+1.958	+5.25	+5.55	+1.680	+1.776	.....	.....	.....	.....
	0.266	.....	.....	.....	.....	.....	+9.45	.....	+2.513	.....	.....	.....	.....	.....	.....	.....	.....
	0.258	.....	.....	.....	.....	+9.07	.....	+2.340	.....	.....	.....	.....	.....	.....	.....	.....	.....
11	0.108	−23.62	−21.95	−3.472	−3.284	+8.55	+8.90	+0.923	+0.961	+4.67	+5.00	+0.504	+0.540	.....	.....	.....	.....

TABLE 14.—COMPUTATIONS FOR MOMENTS IN ARCH A_1 OF FIVE-SPAN SYSTEM, UNYIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.087	+ 3.96	+ 4.37	+0.344	+0.380	+ 0.87	+ 1.25	+0.075	+0.108	−0.85	−0.50	−0.073	−0.043	− 1.75	− 1.42	−0.152	−0.123
2	0.238	+ 4.00	+ 4.41	+0.952	+1.049	+ 0.89	+ 1.27	+0.211	+0.302	−0.84	−0.49	−0.199	−0.116	− 1.77	− 1.44	−0.421	−0.342
3	0.392	+ 4.08	+ 4.49	+1.599	+1.760	+ 0.96	+ 1.31	+0.376	+0.483	−0.83	−0.48	−0.325	−0.188	− 1.79	− 1.46	−0.701	−0.572
4	0.502	+ 4.27	+ 4.69	+2.152	+2.363	+ 1.04	+ 1.40	+0.524	+0.705	−0.82	−0.47	−0.413	−0.236	− 1.87	− 1.54	−0.942	−0.776
5	0.513	+ 4.36	+ 4.77	+2·236	+2.447	+ 1.08	+ 1.46	+0.554	+0.748	−0.81	−0.46	−0.415	−0.235	− 1.93	− 1.60	−0.990	−0.820
6	0.500	+ 4.52	+ 4.92	+2.278	+2.479	+ 1.15	+ 1.52	+0.579	+0.766	−0.80	−0.44	−0.403	−0.221	− 2.00	− 1.67	−1.008	−0.841
7	0.483	+ 4.71	+ 5.12	+2.274	+2.472	+ 1.29	+ 1.63	+0.623	+0.787	−0.78	−0.42	−0.376	−0.202	− 2.09	− 1.76	−1.009	−0.850
8	0.413	+ 5.00	+ 5.40	+2.065	+2.230	+ 1.39	+ 1.75	+0.574	+0.722	−0.75	−0.39	−0.315	−0.163	− 2.23	− 1.90	−0.920	−0.784
9	0.258	+ 6.08	+ 6.48	+1.568	+1.684	+ 1.94	+ 2.29	+0.504	+0.595	−0.71	−0.36	−0.184	−0.093	− 2.74	− 2.41	−0.706	−0.621
10	0.094	+ 9.08	+ 9.44	+0.839	+0.925	+ 3.48	+ 3.81	+0.341	+0.373	−0.52	−0.16	−0.050	−0.015	− 4.09	− 3.76	−0.324	−0.353
11	0.009	+37.44	+37.53	+0.374	+0.375	+17.70	+17.82	+0.177	+0.178	+1.18	+1.44	+0.011	+0.014	−17.22	−16.94	−0.154	−0.152

Position of load.	H.	Section V.				Section VI.				Section VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.087	− 1.44	− 1.08	−0.125	−0.093	− 0.18	+ 0.16	−0.015	+0.013	+ 2.30	+ 2.72	+0.200	+0.236
2	0.238	− 1.50	− 1.14	−0.357	−0.271	− 0.29	+ 0.06	−0.069	+0.014	+ 2.16	+ 2.58	+0.514	+0.614
3	0.392	− 1.56	− 1.21	−0.611	−0.474	− 0.41	− 0.05	−0.160	−0.019	+ 2.06	+ 2.44	+0.807	+0.956
4	0.502	− 1.75	− 1.40	−0.882	−0.705	− 0.65	− 0.29	−0.327	−0.146	+ 1.69	+ 2.10	+0.851	+1.058
5	0.513	− 1.85	− 1.50	−0.949	−0.769	− 0.82	− 0.44	−0.420	−0.225	+ 1.50	+ 1.91	+0.769	+0.979
6	0.500	− 2.00	− 1.64	−1.008	−0.826	− 1.02	− 0.65	−0.514	−0.327	+ 1.20	+ 1.60	+0.604	+0.806
7	0.483	− 2.21	− 1.85	−1.067	−0.893	− 1.31	− 0.94	−0.632	−0.454	+ 0.81	+ 1.21	+0.391	+0.584
8	0.413	− 2.48	− 2.12	−1.024	−0.875	− 1.73	− 1.35	−0.714	−0.557	+ 0.25	+ 0.67	+0.103	+0.276
9	0.258	− 3.54	− 3.18	−0.917	−0.820	− 3.25	− 2.94	−0.838	−0.758	− 1.83	− 1.38	−0.472	−0.356
10	0.094	− 6.50	− 6.12	−0.611	−0.575	− 7.50	− 7.12	−0.705	−0.669	− 7.56	− 7.08	−0.710	−0.678
11	0.009	−34.60	−34.18	−0.311	−0.307	−48.31	−47.74	−0.434	−0.429	−62.78	−62.06	−0.565	−0.558

TABLE 15.—COMPUTATIONS FOR MOMENTS IN ARCH A_0 OF FIVE-SPAN SYSTEM, UNYIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.086	−3.18	−3.62	−0.273	−0.311	−0.50	−0.87	−0.043	−0.074	−0.87	−0.52	−0.074	−0.044	−1.37	−1.04	−0.117	−0.089
2	0.237	−3.17	−3.61	−0.751	−0.855	−0.50	−0.87	−0.118	−0.206	−0.87	−0.52	−0.206	−0.123	−1.37	−1.04	−0.324	−0.246
3	0.388	−3.16	−3.60	−1.226	−1.396	−0.50	−0.87	−0.194	−0.337	−0.87	−0.52	−0.337	−0.201	−1.36	−1.03	−0.527	−0.399
4	0.502	−3.15	−3.59	−1.581	−1.802	−0.50	−0.86	−0.251	−0.431	−0.88	−0.50	−0.441	−0.266	−1.35	−1.02	−0.677	−0.512
5	0.513	−3.14	−3.58	−1.610	−1.836	−0.50	−0.85	−0.256	−0.436	−0.88	−0.53	−0.451	−0.271	−1.34	−1.01	−0.687	−0.518
6	0.504	−3.13	−3.57	−1.577	−1.799	−0.49	−0.84	−0.246	−0.423	−0.89	−0.54	−0.448	−0.272	−1.33	−1.00	−0.670	−0.504
7	0.489	−3.12	−3.55	−1.525	−1.735	−0.48	−0.83	−0.234	−0.405	−0.90	−0.54	−0.440	−0.264	−1.32	−0.99	−0.645	−0.484
8	0.423	−3.06	−3.48	−1.294	−1.472	−0.43	−0.79	−0.181	−0.334	−0.92	−0.55	−0.389	−0.232	−1.31	−0.98	−0.554	−0.414
9	0.273	−2.90	−3.31	−0.791	−0.903	−0.34	−0.71	−0.092	−0.193	−0.94	−0.56	−0.256	−0.152	−1.27	−0.94	−0.346	−0.256
10	0.108	−2.62	−3.04	−0.282	−0.328	−0.19	−0.56	−0.020	−0.060	−0.96	−0.62	−0.103	−0.066	−1.18	−0.85	−0.127	−0.091
11	0.015	−1.20	−1.65	−0.018	−0.024	−0.54	−0.18	−0.008	−0.002	−1.14	−0.79	−0.017	−0.011	−0.69	−0.36	−0.010	−0.005

Position of load.	H.	Section V.				Section VI.				Section VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.086	−0.68	−0.33	−0.058	−0.028	−0.87	−1.25	−0.074	−0.107	−3.75	−4.14	−0.322	−0.356
2	0.237	−0.67	−0.32	−0.158	−0.075	−0.87	−1.25	−0.206	−0.296	−3.75	−4.15	−0.888	−0.983
3	0.388	−0.66	−0.31	−0.256	−0.120	−0.88	−1.25	−0.341	−0.485	−3.76	−4.16	−1.458	−1.614
4	0.502	−0.65	−0.30	−0.326	−0.150	−0.89	−1.26	−0.446	−0.632	−3.77	−4.17	−1.892	−2.093
5	0.513	−0.64	−0.29	−0.328	−0.148	−0.90	−1.27	−0.461	−0.651	−3.78	−4.18	−1.939	−2.144
6	0.504	−0.63	−0.28	−0.317	−0.141	−0.91	−1.28	−0.458	−0.645	−3.79	−4.19	−1.910	−2.111
7	0.489	−0.61	−0.27	−0.298	−0.132	−0.92	−1.29	−0.449	−0.630	−3.80	−4.21	−1.858	−2.058
8	0.423	−0.54	−0.20	−0.228	−0.084	−1.04	−1.37	−0.439	−0.578	−3.98	−4.35	−1.662	−1.840
9	0.273	−0.43	−0.08	−0.117	−0.021	−1.23	−1.58	−0.335	−0.431	−4.20	−4.61	−1.146	−1.258
10	0.108	−0.20	−0.12	−0.021	−0.012	−1.54	−1.92	−0.166	−0.207	−4.64	−5.06	−0.501	−0.546
11	0.015	−0.96	−1.29	−0.014	−0.019	−3.29	−3.64	−0.049	−0.054	−7.04	−7.42	−0.105	−0.111

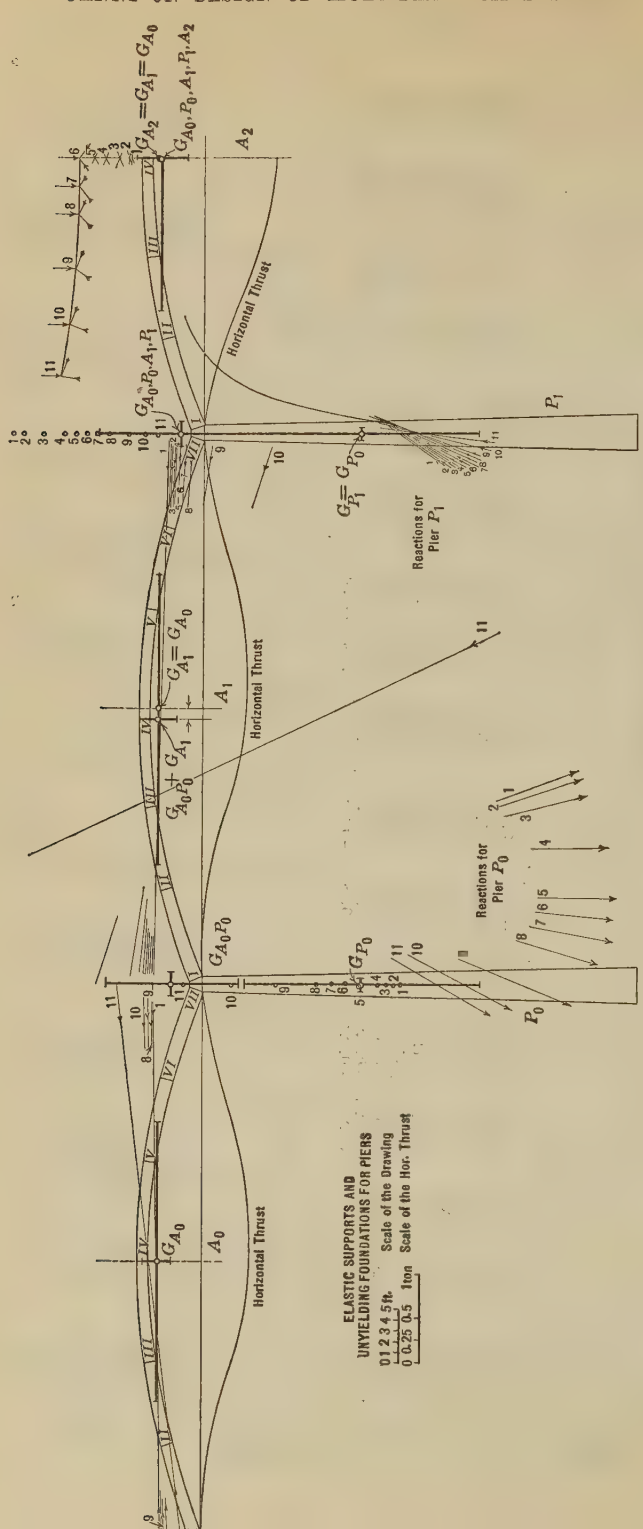

FIG. 66.—ELASTIC SUPPORTS AND UNYIELDING FOUNDATIONS FOR PIERS.

It is worthy of note that, while the forces acting on Pier P_1 are of compression, those acting on Pier P_0 are of tension, that is, at Pier P_0 there is uplift.

Therefore, the horizontal thrust for Arch A_0 is larger than that for Arch A_1. This result is shown practically by comparing the values of horizontal thrusts recorded in Tables 14 and 15.

Analysis of Five-Arch System.—Yielding Foundations.—Assume now that, having the same system of five arches, the footings of both Piers P_1 and P_2 yield; the effect that this yielding will have on all arches of this system is required.

For simplicity the yielding soil is assumed to be the same for both piers. This assumption makes the diagram symmetrical with respect to the center vertical line of the elastic system, reducing considerably the computations involved. For unsymmetrical yielding the method is obviously the same, only longer, for in this case it is necessary also to determine the ellipse of the right joint which would not be the same as that on the left.

In order to take care of this yielding, it is necessary to add to the ellipse of elasticity of the pier another ellipse having the horizontal axis only, for the horizontal displacement of the footing is disregarded.

Having to deal with a bearing surface of 3 ft. 4 in. by 1 ft., assume the nature of the soil to be such as to yield 1 in. vertically under a pressure of 15 tons per sq. ft. The elastic weight of this surface is given by:*

$$\phi_0 = \frac{1}{1\,105\,642} \times 288\,000\,000 = 260.48$$

in which, $E = 1$, as has been assumed for the elastic weights of the whole structure.

Therefore, $\rho = \sqrt{\frac{3.07}{3.33}} = 0.96$ ft. = horizontal semi-axis of the ellipse of elasticity of the soil.

Ellipse of Pier P_1.—Having found the value of ρ for this particular ellipse, it is possible to determine the ellipse of elasticity of Pier P_1 as follows:

Elastic weight of the pier itself	74.61
" " " " soil	260.48
Total	335.09
Sum of moment of inertia of the elastic weights of the pier proper	18.836
Moment of inertia of the soil surface's elastic weight, 260.48×0.96^2	240.058
Total	258.894
Sum of the static moment of the elastic weights with respect to the top section of the pier	11 510.54

* "Reinforced Concrete Construction," by George A. Hool, Vol. III.

Distance of the elastic center from this section,

$$\frac{11510.54}{335.09} = 34.35 \text{ ft.}$$

$$\text{Minor axis, } \rho' = \sqrt{\frac{258.89}{335.09}} = 0.87 \text{ ft.}$$

$$\text{Major axis, } \rho = \sqrt{\frac{20 \times 10 \times 23.15}{33.50}} = 11.74 \text{ ft.}$$

Ellipse of the Joint Between $G_{A_0 P_0 A_1}$ and G_{P_1}'.—The necessary results for this joint are given herewith. The methods of making these computations follow those already explained.

$$d_1 = 37.78 \text{ ft.}; \ \rho = 5.98 \text{ ft.}; \ G'_{A_0 P_0 A_1 P_1} = 25.39$$
$$d_1' = 0.09 \text{ ft.}; \ \rho' = 3.14 \text{ ft.}$$

Analysis of Arch A_2.—This analysis is the same in every detail as those previously shown. Fig. 67 gives the three corresponding diagrams. The elastic quantities are given in Table 16. The resulting moments for Sections I, II, III, and IV have been computed and noted in Table 17.

On Fig. 68 are shown the graphical operations necessary for the determination of the forces acting on Arches A_1 and A_0 when Arch A_2 is being loaded and when, furthermore, its supports are built on yielding soil of the assumed modulus of elasticity. The reactions on Piers P_1 and P_0 are also shown. The computations for Arches A_1 and A_0 are given in Tables 18 and 19. A pictorial view of the structures treated in the paper and this discussion is shown in Fig. 69 in a condensed form. The four hypotheses noted are those referred to more in detail in subsequent diagrams.

All the moments obtained in all four hypotheses contemplated and recorded in Figs. 66 and 68 have been plotted together for the sake of comparison. The results (Figs. 70, 71, and 72) show clearly what had been forecast, that the variation of moments for the various sections of Arch A_2 is large only when Arch A_2 passes from fixed supports to elastic supports, constituted each by an arch and a pier, and that the further addition of an arch and pier on each side does not cause any important variation in the moments of the arch.

Interpretation of Results

It is to be noted also that by increasing further the number of arches and piers on each side of Arch A_2, the difference in the moments, between two consecutive increases, will be smaller than the preceding one.

It is interesting also to see the behavior of the system of five arches when the two piers yield according to the assumption made. From an inspection of Figs. 66 and 68, it is seen that the angle formed by Reactions 1 and 11 (extreme reactions) diminishes rapidly.

Considering the various constructions for the determination of the positions of these reactions, it is safe to conclude that in a long series of arches these reactions will tend to crowd together very rapidly, so that there will not be any appreciable error by considering the direction and position of one reaction as that of all reactions for an arch far removed from the center

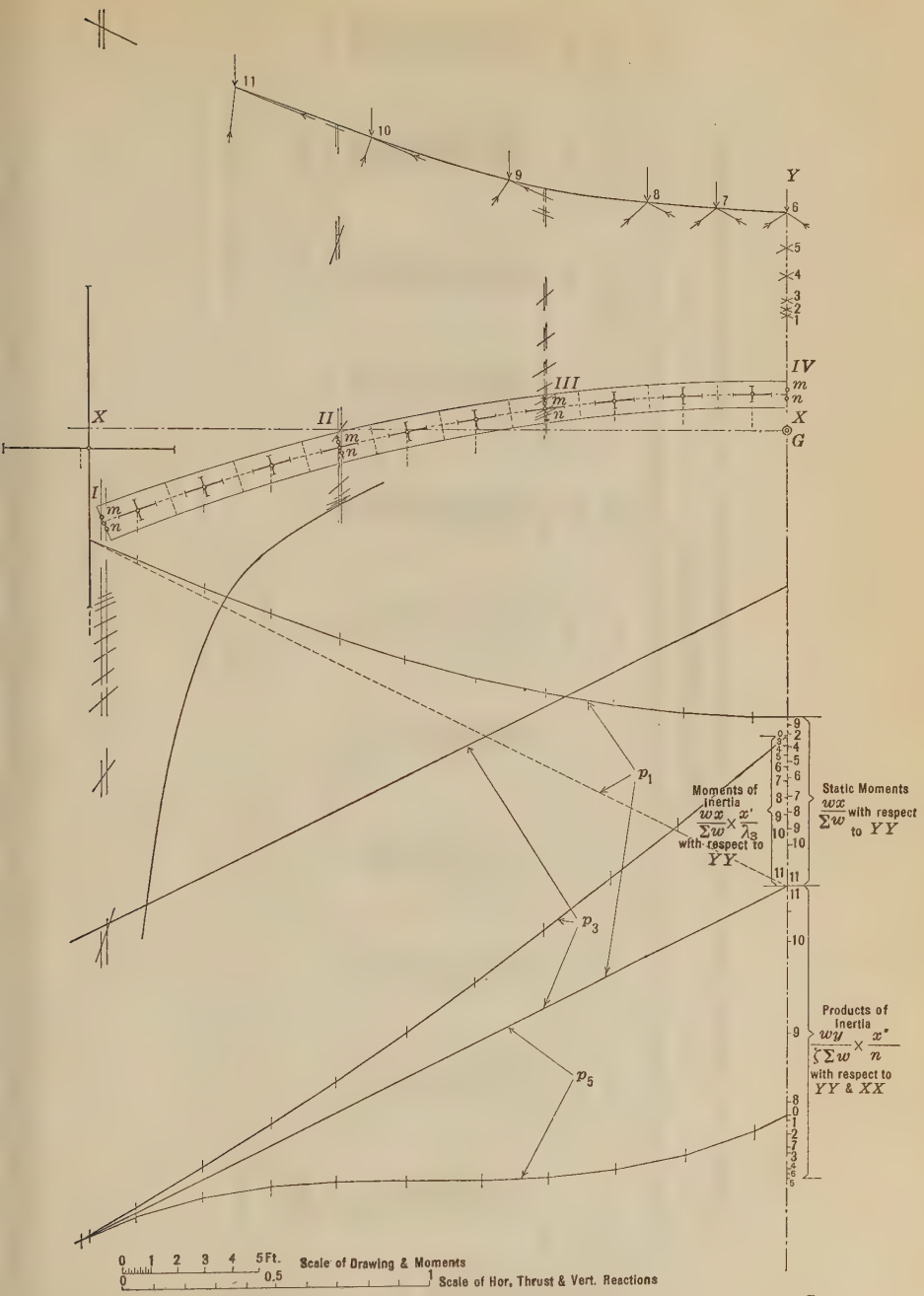

FIG. 67.—DESIGN OF A MULTIPLE-ARCH SYSTEM. ANALYSIS OF ARCH A_2 AS THE CENTER ARCH OF A FIVE-ARCH SYSTEM AND PIERS P_1 AND P_2 BUILT ON YIELDING SOIL.

TABLE 16.—Computations of Elastic Quantities for Center Arch A_2 of Five-Arch System, Yielding Foundations.

Position of load.	w. (1)	x. (2)	y. (3)	x'. (4)	y'. (5)	x''. (6)	$\frac{w\,x}{\Sigma\,w}$. (7)	$\frac{w\,y}{\zeta\,\Sigma\,w}$. (8)	$\frac{w\,x}{\Sigma\,w}\cdot\frac{x'}{\lambda_3}$. (9)	$\frac{w\,y}{\zeta\,\Sigma\,w}\cdot\frac{y'}{v}$. (10)	$\frac{w\,y}{\zeta\,\Sigma\,w}\cdot\frac{x''}{n}$. (11)
1	2.55	0.64	0.67	0.85	0.72	0.63	0.038	0.282	0.003	0.036	0.09
2	2.41	1.94	0.62	2.03	0.67	1.91	0.112	0.246	0.022	0.029	0.26
3	2.28	3.23	0.51	3.29	0.59	3.21	0.173	0.192	0.056	0.020	0.34
4	2.15	4.51	0.36	4.56	0.44	4.47	0.228	0.128	0.103	0.009	0.32
5	2.03	5.79	0.17	5.84	0.34	5.70	0.277	0.056	0.161	0.003	0.17
6	1.93	7.06	0.07	7.10	0.57	7.40	0.321	0.022	0.227	0.002	0.09
7	1.72	8.31	0.37	8.31	0.47	8.39	0.337	0.105	0.280	0.011	0.49
8	1.38	9.57	0.71	9.57	0.77	9.60	0.311	0.161	0.297	0.021	0.85
9	1.19	10.81	1.10	10.81	1.14	10.83	0.303	0.216	0.327	0.043	1.29
10	1.03	12.03	1.53	12.03	1.54	12.04	0.292	0.260	0.351	0.070	1.74
11	2.53	12.88	0.35	13.04	25.54	12.88	0.768	0.146	1.001	0.659	1.04
....	21.20 2	.....	.....	.....	.....	.....	.....	.....	2.828 2	0.903 2	.....
$\Sigma\,w =$	42.40	.....	.....	.....	.....	.....	.....	.....	$v = 5.656$ in.	$n = 1.806$ in.	.....

TABLE 17.—Computations for Moments in Arch A_2 (Center Span of a Five-Span System), with Two Yielding Foundations.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.097	— 2.90	— 2.46	—0.281	—0.238	— 2.10	— 1.78	—0.203	—0.172	—0.43	—0.05	— 0.041	—0.004	—2.75	—3.08	—0.266	—0.298
2	0.283	— 3.00	— 2.50	—0.849	—0.707	— 2.12	— 1.79	—0.599	—0.506	—0.33	—0.08	—0.093	—0.022	—2.95	—3.28	—0.834	—0.928
3	0.508	— 3.15	— 2.65	—1.600	—1.346	— 2.14	— 1.80	—1.087	—0.914	—0.20	—0.18	—0.101	—0.091	—3.17	—3.50	—1.610	—1.778
4	0.690	— 3.80	— 3.30	—2.622	—2.277	— 2.17	— 1.81	—1.497	—1.248	—0.25	—0.62	—0.172	—0.427	—4.30	—4.63	—2.967	—3.194
5	0.743	— 4.45	— 3.95	—3.306	—2.934	— 2.20	— 1.82	—1.634	—1.352	—0.65	—1.03	—0.489	—0.765	—5.25	—5.58	—3.900	—4.145
6	0.761	— 5.10	— 4.60	—3.881	—3.500	— 2.25	— 1.83	—1.712	—1.392	—1.28	—1.65	—0.974	—1.255	—6.60	—6.93	—5.022	—5.273
7	0.743	— 5.90	— 5.40	—4.383	—4.012	— 2.05	— 1.65	—1.523	—1.225	—2.38	—2.78	—1.768	—2.065	.....	.....	.....	.....
8	0.690	— 6.75	— 6.20	—4.657	—4.278	— 1.55	— 1.12	—1.069	—0.772	—4.00	—4.40	—2.760	—3.036	.....	.....	.....	.....
.....	0.566	.....	.....	.....	.....	.....	.....	.....	.....	.....	—8.40	.....	—4.754	.....	.....	.....	.....
.....	0.563	.....	.....	.....	.....	.....	.....	.....	.....	—8.05	.....	—4.532	.....	.....	.....	.....	.....
9	0.508	— 9.65	— 9.05	—4.902	—4.597	— 0.40	— 0.90	—0.203	—0.457	—7.75	—8.07	—3.937	—4.099	.....	.....	.....	.....
10	0.283	—16.00	—15.05	—4.528	—4.259	— 7.40	— 8.10	—2.094	—2.292	—7.20	—7.50	—2.037	—2.122	.....	.....	.....	.....
.....	0.234	.....	.....	.....	.....	.....	—12.15	.....	—2.843	.....	.....	.....	.....	.....	.....	.....	.....
.....	0.230	.....	.....	.....	.....	—11.80	.....	—2.714	.....	.....	.....	.....	.....	.....	.....	.....	.....
11	0.097	—28.60	—26.65	—2.774	—2.585	—11.70	—12.00	—1.134	—1.164	—7.04	—7.38	—0.682	—0.715	.....	.....	.....	.....

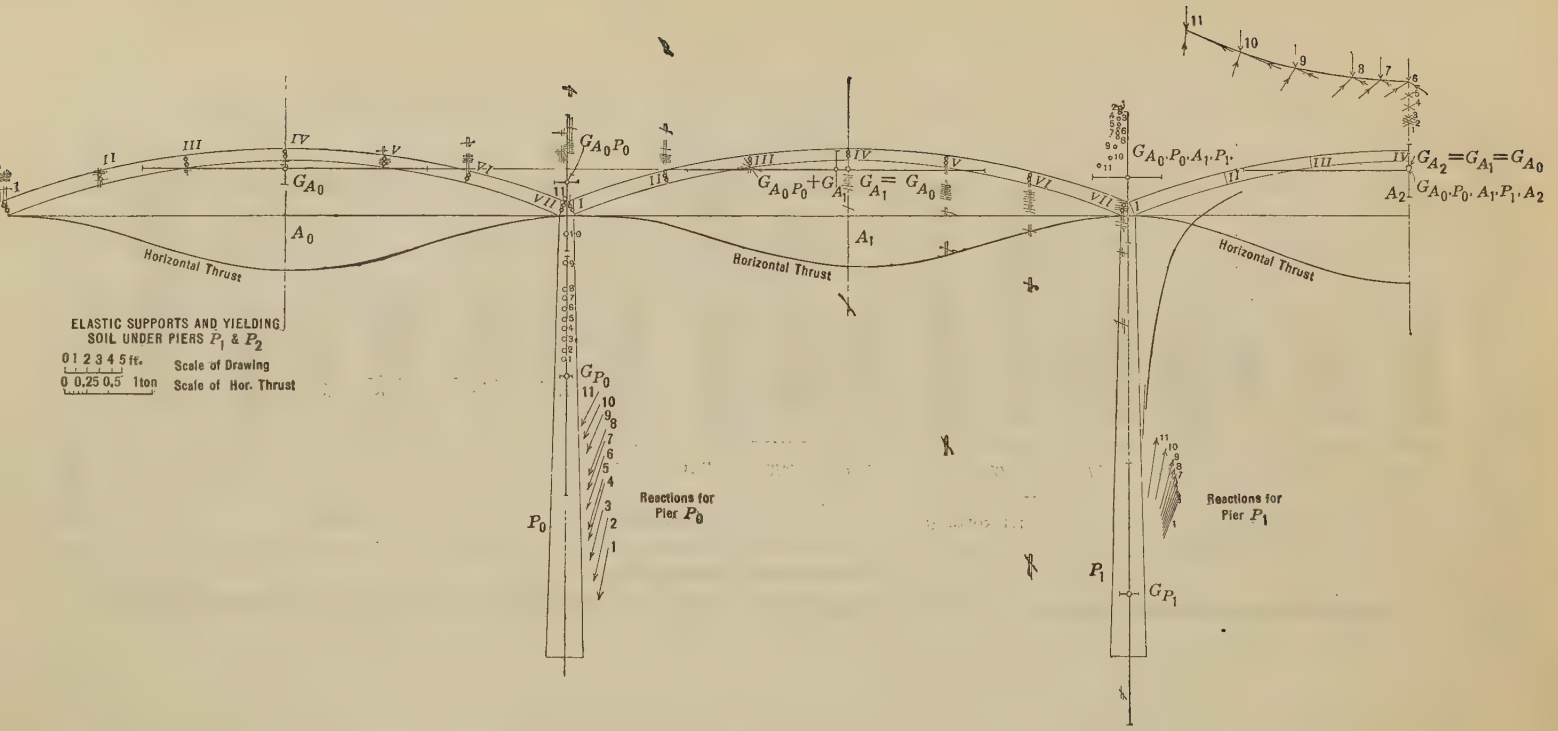
ELASTIC SUPPORTS AND YIELDING SOIL UNDER PIERS P_1 & P_2
0 1 2 3 4 5 ft. Scale of Drawing
0 0.25 0.5 1 ton Scale of Hor. Thrust
G_{A_0}
A_0
Horizontal Thrust
$G_{A_0 P_0}$
$G_{A_0 P_0} + G_{A_1}$
$G_{A_1} = G_{A_0}$
A_1
Horizontal Thrust
$G_{A_0 P_0 A_1 P_1}$
$G_{A_2} = G_{A_1} = G_{A_0}$
$G_{A_0 P_0 A_1 P_1 A_2}$
A_2
Horizontal Thrust
G_{P_0}
P_0
Reactions for Pier P_0
P_1
G_{P_1}
Reactions for Pier P_1

FIG. 68

of the system, due to the load of the central arch, and that this direction is very nearly horizontal. In the presence of these results it seems that the simplification proposed by Professor Guidi is fully justified.

In presenting the paper the writer had no idea of giving a complete analysis of a multiple-arch system. His only aim was to emphasize the fact that, having to deal with, say, a system of ten arches in which there are no abutment piers, it is possible to reduce the work of analysis considerably.

There are other structural elements in a system like this, as, for instance, piers, arches, etc., which are subject to proper analysis, whether or not the suggested simplification is adopted for the middle arches.

The writer will outline in a few words the method to be followed in the analysis of the end arches. In order to find the moments, etc., for Arch A_4, for instance, it is necessary to determine the following ellipses: G_{A_0}; G_{P_0}; $G_{A_0P_0}$; $G_{A_0P_0A_1}$; $G_{A_0P_0A_1P_1}$; $G_{A_0P_0A_1P_1A_2}$; $G_{A_0P_0A_1P_1A_2P_2}$; $G_{A_0P_0A_1P_1A_2P_2A_3}$; and $G_{A_0P_0A_1P_1A_2P_2A_3P_3}$. Then this last ellipse must be considered in its correct position, as an ideal voussoir on the left of Arch A_4, and Arch A_4 analyzed as has been shown previously.

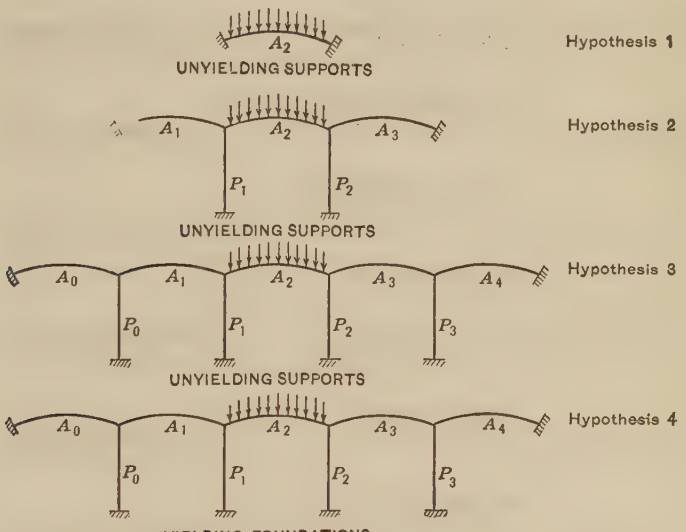

FIG. 69.

It is to be noted, however, that as the elastic system is no longer symmetrical, in this case, with respect to the center line of the geometrical arch, all constructions for the determination of the polygons, p_1, p_3, and p_5, must be done for the whole arch (see Appendix, Paragraph 27 *et seq.*)* and cannot be limited to the half arch as has been done in the preceding cases. Fig. 73 shows how the analysis of such an arch is to be started. This method is strictly theoretical and hence the work involved is, generally speaking, not commensurate with the usual practical results.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1176 *et seq.*

TABLE 18.—COMPUTATIONS FOR MOMENTS IN ARCH A_1 OF FIVE-SPAN SYSTEM WHEN PIERS P_1 AND P_2 REST ON YIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.083	— 4.80	— 5.22	—0.398	—0.423	— 1.27	— 1.62	—0.105	—0.134	—0.87	—0.50	—0.072	—0.041	— 2.20	— 1.87	—0.182	—0.155
2	0.230	— 4.90	— 5.33	—1.127	—1.225	— 1.35	— 1.68	—0.310	—0.386	—0.86	—0.49	—0.197	—0.112	— 2.25	— 1.92	—0.517	—0.441
3	0.414	— 5.08	— 5.50	—2.103	—2.277	— 1.42	— 1.80	—0.587	—0.745	—0.85	—0.48	—0.351	—0.198	— 2.32	— 1.99	—0.960	—0.823
4	0.553	— 5.20	— 5.62	—2.875	—3.107	— 1.48	— 1.85	—0.818	—1.023	—0.84	—0.47	—0.464	—0.259	— 2.39	— 2.03	—1.321	—1.139
5	0.576	— 5.32	— 5.73	—3.064	—3.300	— 1.55	— 1.90	—0.892	—1.094	—0.80	—0.46	—0.478	—0.264	— 2.47	— 2.14	—1.422	—1.232
6	0.578	— 5.55	— 5.98	—3.207	—3.456	— 1.68	— 2.02	—0.971	—1.167	—0.82	—0.45	—0.473	—0.260	— 2.58	— 2.25	—1.491	—1.300
7	0.551	— 5.80	— 6.23	—3.195	—3.432	— 1.80	— 2.13	—0.991	—1.173	—0.81	—0.44	—0.446	—0.242	— 2.70	— 2.37	—1.487	—1.250
8	0.502	— 6.00	— 6.40	—3.012	—3.212	— 1.90	— 2.23	—0.953	—1.119	—0.80	—0.43	—0.401	—0.215	— 2.80	— 2.47	—1.405	—1.239
9	0.340	— 7.23	— 7.62	—2.458	—2.590	— 2.50	— 2.83	—0.850	—0.962	—0.78	—0.40	—0.265	—0.136	— 3.38	— 3.05	—1.149	—1.037
10	0.146	—10.50	—10.85	—1.535	—1.584	— 4.10	— 4.40	—0.598	—0.642	—0.65	—0.32	—0.094	—0.046	— 5.05	— 4.72	—0.737	—0.689
11	0.027	—26.80	—27.00	—0.723	—0.729	—11.90	—12.10	—0.321	—0.326	—0.60	—0.28	—0.016	—0.007	—13.79	—13.46	—0.372	—0.363

Position of load.	H.	Section V.				Section VI.				Section VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	—0.083	— 2.35	— 2.00	—0.195	—0.166	— 1.55	— 1.11	—0.128	—0.092	— 0.50	— 0.96	—0.041	—0.079
2	—0.230	— 2.47	— 2.10	—0.568	—0.483	— 1.70	— 1.25	—0.391	—0.287	— 0.38	— 0.80	—0.087	—0.184
3	—0.414	— 2.63	— 2.27	—1.088	—0.939	— 1.95	— 1.53	—0.807	—0.633	0	— 0.45	0	—0.186
4	—0.553	— 2.75	— 2.40	—1.520	—1.327	— 2.11	— 1.68	—1.166	—0.929	— 0.26	— 0.20	—0.143	—0.110
5	—0.576	— 2.90	— 2.56	—1.670	—1.474	— 2.32	— 1.90	—1.336	—1.094	— 0.50	— 0.08	—0.288	—0.046
6	—0.578	— 3.12	— 2.77	—1.803	—1.601	— 2.65	— 2.23	—1.531	—1.288	— 0.98	— 0.50	—0.566	—0.289
7	—0.551	— 3.43	— 3.07	—1.889	—1.691	— 3.08	— 2.65	—1.697	—1.460	— 1.55	— 1.05	—0.854	—0.578
8	—0.502	— 3.62	— 3.24	—1.817	—1.626	— 3.30	— 2.88	—1.656	—1.445	— 1.85	— 1.38	—0.928	—0.692
9	—0.340	— 4.80	— 4.40	—1.632	—1.496	— 5.07	— 4.63	—1.723	—1.574	— 4.23	— 3.78	—1.488	—1.285
10	—0.146	— 8.22	— 7.88	—1.200	—1.150	—10.00	— 9.55	—1.460	—1.394	—10.80	—10.30	—1.576	—1.503
11	—0.027	—26.00	—25.60	—0.706	—0.691	—35.33	—34.80	—0.953	—0.939	—44.68	—43.95	—1.206	—1.186

TABLE 19.—Computations for Moments in Arch A_0 of Five-Span System when Piers P_1 and P_2 Rest on Yielding Foundations.

Position of load.	$H.$	Section I.				Section II.				Section III.				Section IV.			
		$h_m.$	$h_n.$	$M_m.$	$M_n.$	$h_m.$	$h_n.$	$M_m.$	$M_n.$	$h_m.$	$h_n.$	$M_m.$	$M_n.$	$h_m.$	$h_n.$	$M_m.$	$M_n.$
1	0.087	−2.96	−3.40	−0.257	−0.295	−0.40	−0.80	−0.084	−0.069	−0.88	−0.54	−0.076	−0.046	−1.30	−0.97	−0.113	−0.084
2	0.238	−2.90	−3.35	−0.690	−0.797	−0.37	−0.77	−0.088	−0.183	−0.89	−0.54	−0.211	−0.128	−1.28	−0.95	−0.304	−0.226
3	0.435	−2.84	−3.30	−1.235	−1.435	−0.34	−0.74	−0.147	−0.321	−0.90	−0.54	−0.391	−0.234	−1.26	−0.93	−0.548	−0.404
4	0.584	−2.78	−3.25	−1.623	−1.898	−0.31	−0.71	−0.181	−0.414	−0.91	−0.54	−0.531	−0.315	−1.24	−0.91	−0.724	−0.531
5	0.608	−2.72	−3.20	−1.653	−1.945	−0.28	−0.68	−0.170	−0.413	−0.92	−0.55	−0.559	−0.331	−1.22	−0.89	−0.741	−0.541
6	0.617	−2.66	−3.15	−1.641	−1.943	−0.25	−0.65	−0.154	−0.401	−0.93	−0.56	−0.604	−0.345	−1.20	−0.87	−0.740	−0.536
7	0.593	−2.60	−3.10	−1.541	−1.838	−0.22	−0.62	−0.130	−0.367	−0.94	−0.57	−0.557	−0.338	−1.18	−0.85	−0.699	−0.504
8	0.542	−2.54	−3.05	−1.376	−1.653	−0.18	−0.58	−0.097	−0.314	−0.95	−0.58	−0.514	−0.314	−1.16	−0.83	−0.628	−0.449
9	0.380	−2.48	−3.00	−0.942	−1.140	−0.13	−0.54	−0.049	−0.205	−0.96	−0.59	−0.364	−0.224	−1.14	−0.81	−0.433	−0.307
10	0.178	−2.40	−2.83	−0.427	−0.503	−0.08	−0.50	−0.014	−0.089	−0.97	−0.60	−0.172	−0.106	−1.12	−0.79	−0.199	−0.140
11	0.046	−1.50	−1.95	−0.069	−0.089	−0.40	−0.02	−0.018	−0.009	−1.10	−0.74	−0.050	−0.034	−0.79	−0.46	−0.036	−0.021

Position of load.	$H.$	Section V.				Section VI.				Section VII.			
		$h_m.$	$h_n.$	$M_m.$	$M_n.$	$h_m.$	$h_n.$	$M_m.$	$M_n.$	$h_m.$	$h_n.$	$M_m.$	$M_n.$
1	0.087	−0.50	−0.15	−0.043	−0.013	−1.15	−1.54	−0.100	−0.133	−4.12	−4.55	−0.358	−0.421
2	0.238	−0.46	−0.11	−0.109	−0.026	−1.22	−1.61	−0.290	−0.383	−4.22	−4.65	−1.004	−1.106
3	0.435	−0.42	−0.07	−0.182	−0.030	−1.29	−1.68	−0.561	−0.730	−4.32	−4.75	−1.879	−2.066
4	0.584	−0.38	−0.03	−0.221	−0.017	−1.36	−1.75	−0.794	−1.022	−4.42	−4.85	−2.581	−2.832
5	0.608	−0.33	−0.02	−0.200	−0.012	−1.43	−1.82	−0.869	−1.106	−4.52	−4.95	−2.748	−3.009
6	0.617	−0.28	−0.07	−0.172	−0.043	−1.51	−1.89	−0.931	−1.166	−4.62	−5.05	−2.850	−3.115
7	0.593	−0.23	−0.12	−0.136	−0.071	−1.59	−1.96	−0.942	−1.162	−4.73	−5.16	−2.804	−3.059
8	0.542	−0.18	−0.17	−0.097	−0.092	−1.67	−2.04	−0.905	−1.105	−4.84	−5.27	−2.623	−2.856
9	0.380	−0.12	−0.23	−0.045	−0.087	−1.76	−2.12	−0.668	−0.805	−4.96	−5.38	−1.884	−2.044
10	0.178	−0.05	−0.30	−0.008	−0.053	−1.85	−2.22	−0.329	−0.398	−5.08	−5.50	−0.904	−0.979
11	0.046	−0.70	−1.05	−0.032	−0.048	−2.95	−3.30	−0.135	−0.151	−6.60	−7.00	−0.303	−0.322

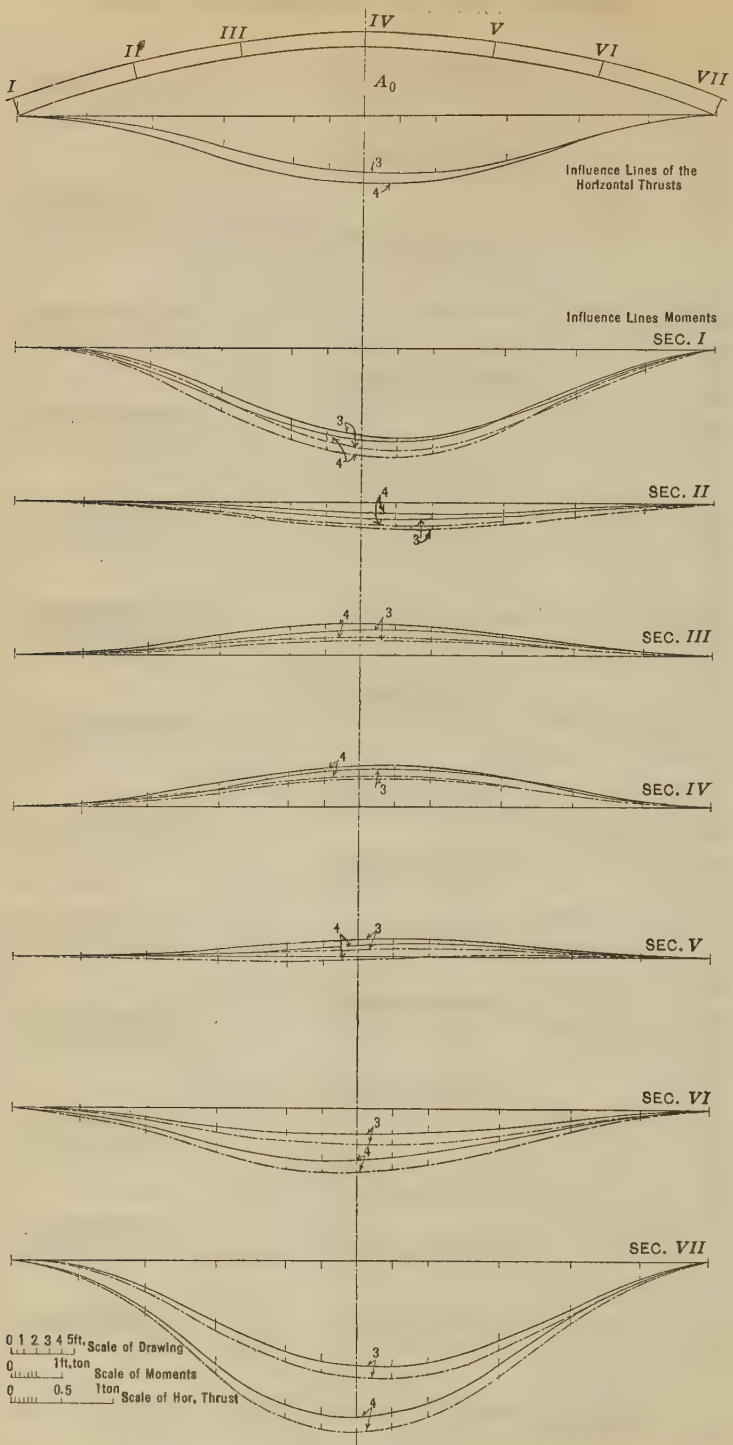

FIG. 70.—INFLUENCE LINES OF MOMENTS AND HORIZONTAL THRUSTS AT VARIOUS SECTIONS OF ARCH A_0.

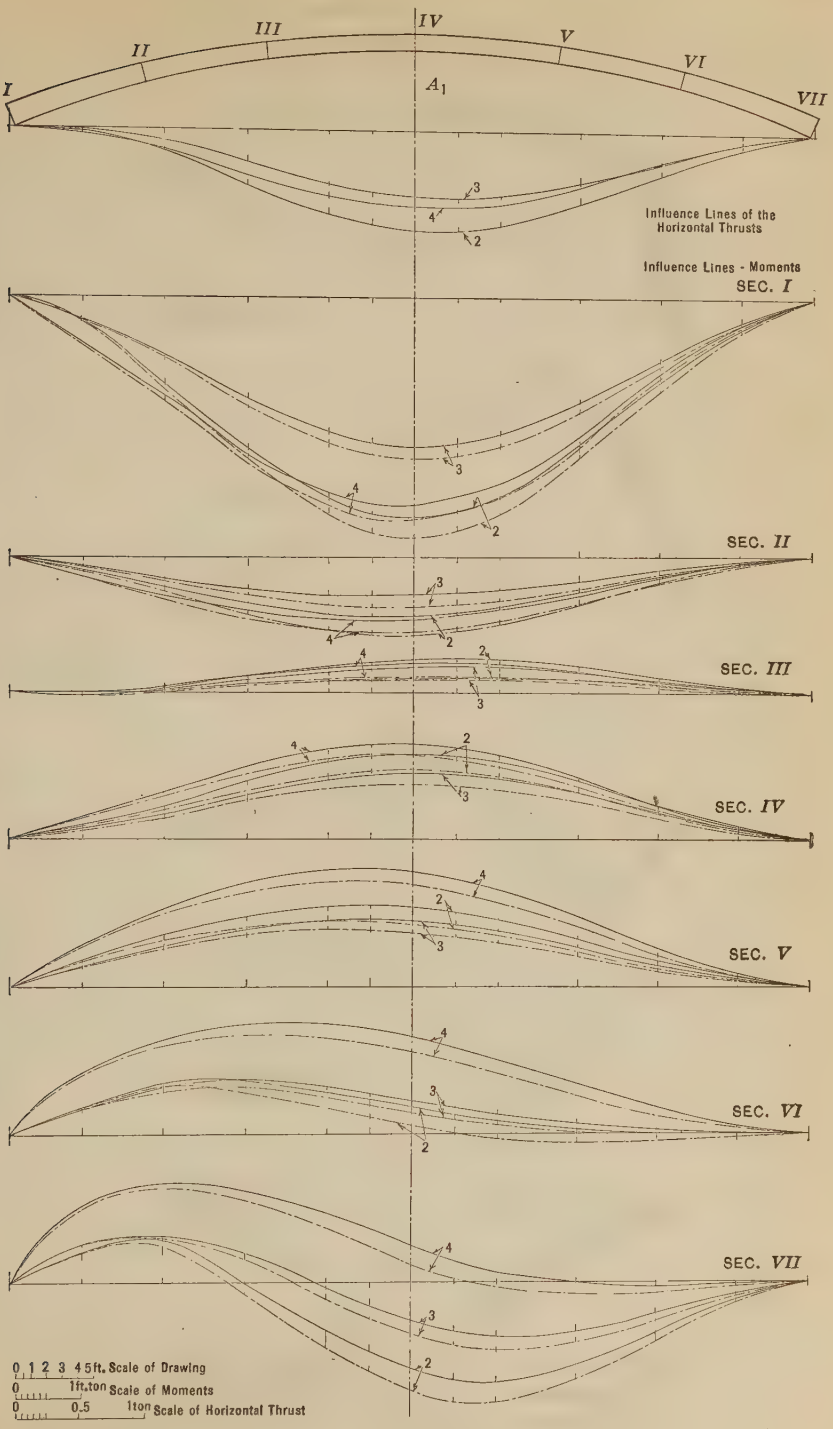

FIG. 71.—INFLUENCE LINES OF MOMENTS AND HORIZONTAL THRUSTS AT VARIOUS SECTIONS OF ARCH A_1.

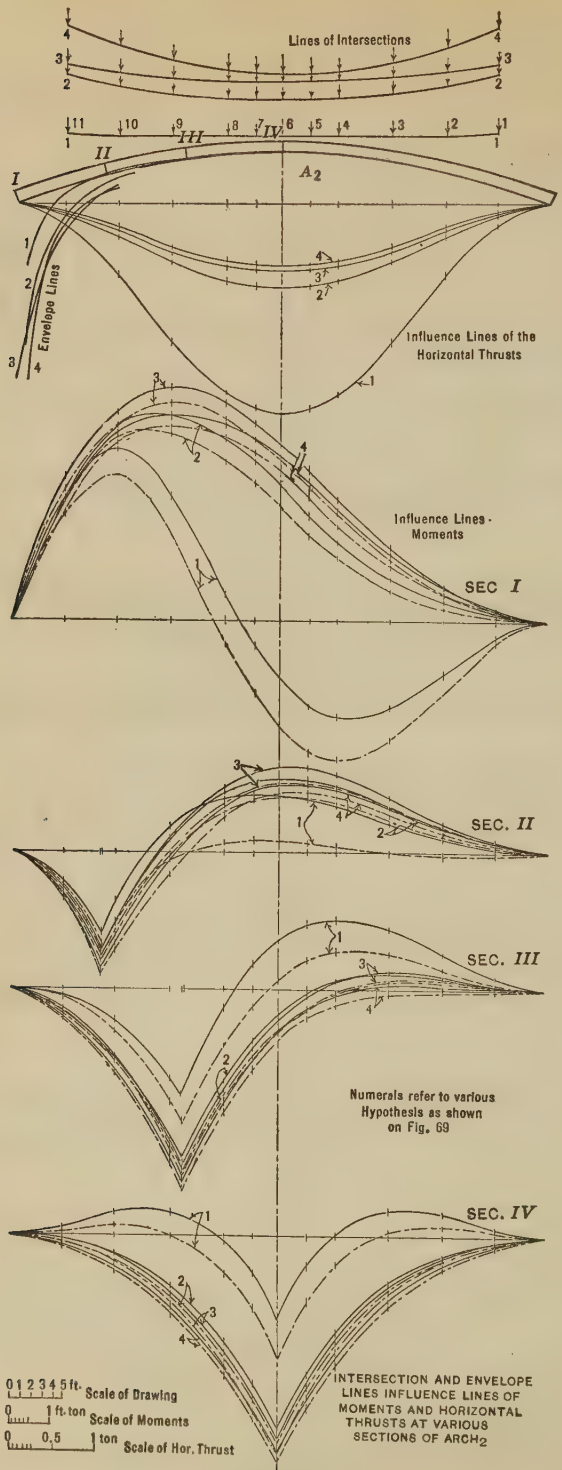
Lines of Intersections
11 10 9 8 7 6 5 4 3 2 1
I
II
III
IV
A_2
Envelope Lines
Influence Lines of the Horizontal Thrusts
Influence Lines - Moments
SEC. I
SEC. II
SEC. III
SEC. IV
Numerals refer to various Hypothesis as shown on Fig. 69
0 1 2 3 4 5 ft. Scale of Drawing
0 1 ft. ton Scale of Moments
0 0.5 1 ton Scale of Hor. Thrust
INTERSECTION AND ENVELOPE LINES INFLUENCE LINES OF MOMENTS AND HORIZONTAL THRUSTS AT VARIOUS SECTIONS OF ARCH 2

FIG. 72.

The abutment for the end span can be taken care of by adding on that side of the arch an ellipse of elasticity, due to the modulus of elasticity of the material constituting that abutment. This ellipse will have one axis only at the end of the arch, just the same as in the case of the pier built on yielding soil. Fig. 73 shows the general arrangement of these ellipses.

From the inspection of the results obtained in the analysis of a multiple-arch system it was evident that, but for exceptionally important cases, the simplification suggested is quite permissible. The same simplification holds good also in the case of end arches. It would curtail very materially the great amount of work required by strictly theoretical treatment of the end arches.

In this case, therefore, Arch A_4 could be analyzed by adding to the various ellipses of elasticity of the voussoirs, the ellipse, $G_{A_3P_3}$ ($G_{A_3P_3} = G_{A_0P_0}$), of the joint between Arch A_3 and Pier P_3, which is readily determined.

Comments on Discussions*

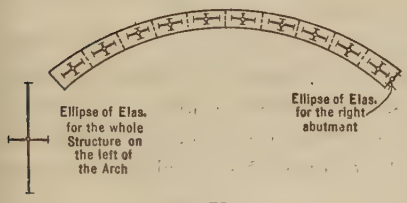

Fig. 73.

Professor Rathbun has discussed this method in comparison with others followed generally in the analysis of a single arch. The further simplifications which may be made in the construction of Polygons p_1, p_2, p_3, p_4, and p_5, here shown, make this method still more rapid, so that the saving of time of one-third to one-fifth, as already pointed out by Professor Rathbun, in the case of a full geometrical construction of the five polygons, becomes still greater.

It is true that the full understanding of this method requires some knowledge of the mathematics not studied in American universities, but this knowledge is not difficult to obtain from books on this subject. Probably, of all geometries, Projective Geometry is the simplest, notwithstanding its "discouraging expressions". The study of this branch of mathematics is not an idle one; such knowledge is very useful for the rapid solution of problems of Descriptive Geometry and, applied to complicated theories, as for instance the one in point, makes its explanation very clear, and its application simple and rapid.

However, this theory can be treated analytically, but its demonstration is long and wearisome, as that for the ellipse of inertia of an area. Several works on this subject which treat this theory, mostly in an analytical way, have already been noted.

Mr. Steinman refers to a more expeditious method which he calls a "direct method". He states:†

"The direct method is as follows: To obtain the moment influence line for any panel point of an arch rib, consider a unit force applied along the

* See *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1183 *et seq.*

† *Loc. cit.*, p. 1186.

anti-polar of that panel point with respect to the central ellipse; the resulting deflection curve of the arch is the required moment influence line. This deflection curve is simply obtained as the funicular polygon of static moments of the elastic weights with reference to the anti-polar".

The neatness and brevity of this statement may, at first, convey a wrong idea of the brevity of this "direct method". In fact, in order to follow this method, as pointed out by Mr. Steinman, it is necessary to apply a force along the anti-polar of that panel point with respect to the central ellipse; verbally, this operation is plain and short enough, but in order to do it, it is necessary first to determine that ellipse; that is, the computation of at least Columns (7), (8), (9), (10), Table 12, must be made, by means of which the central ellipse is fully individualized.

The writer, in showing how the position of the reactions can be determined, has preferred a graphical construction which gives, of course, the same result as when the position of the reaction is determined by the method suggested by Mr. Steinman, with the only difference that the construction preferred by the writer is quicker and simpler. The paper also noted that there are several methods for determining this position.

That the method preferred by the writer is far shorter and simpler than that suggested by Mr. Steinman can be seen by the fact that for each position of the reaction (considered as anti-polar) it is necessary first to determine the corresponding pole (center of instantaneous rotation of the free end of the arch under that assumption of loading), the co-ordinates of which are given by:

$$x = \lambda_3 \frac{\eta_1}{\eta_0}$$

$$y = \xi\, n \frac{\eta_2}{\eta_0}$$

while the method followed by the writer involves one computation for each position. Likewise of the graphical constructions, that in the method preferred by the writer is the simpler. Thus it may be seen that the method suggested by Mr. Steinman is nothing but a part of the standard method, and, therefore, it is incomplete.

The enunciation of the principle of the relation between the line of influence and the deflection diagram, as given by Mr. Steinman, does not seem exact. He states, "every influence line is a deflection diagram", which can be taken to mean that the load diagram, for instance, is a deflection diagram in itself, and this would not be correct.

There are two graphical methods to demonstrate the relation between the load diagram and the corresponding moment diagram. Saviotti* obtained, for the first time, the diagram of moments from the diagram of loads by the method called "graphical integration". He demonstrated how it is possible to obtain the diagrams of shear, of moments, and of tangents, and the deformed elastic axes of a beam, by successive graphical integrations of the diagram

* "Sui metodi grafici d'integrazione," C. Saviotti, *Il Giornale del Genio Civile*, 1882.

of loads. The other method is called the "funicular polygon method" by means of which it is possible to shorten the construction involved in the first method so as to obtain the diagram of moments directly from the diagram of loads.

The writer must confess that, in presenting his paper, he took for granted that it was only necessary for him to show the extent of variation in the moments of an arch in the two hypotheses assumed in the paper, namely, an arch on unyielding supports and an arch on yielding supports; and that the yielding part could be considered as constituted by an arch and a pier on each side. He thought that the extension of the elastic supports to more than an arch and a pier was almost useless, for engineers could forecast that the results would not be changed materially, if they had followed mentally the process of distribution of the forces on the lateral arches.

In reading some of the discussions, including that of Mr. Steinman, he has realized that he may have taken too much for granted, and he willingly has supplied what he thinks was wanted. The writer, however, did not overlook "the possibility (which may upset his conclusions) of the two outer spans being loaded simultaneously". From Plate III,* or Tables 7† and 8,‡ it is seen that the maximum moment at Section I for Arch A_2 is verified when Arches A_1 and A_3 are loaded and Arch A_2 is unloaded, and, therefore, as stated in the paper, the maximum moment at Section I for Arch A_2 would be given by twice the total moment obtained from the diagram for Section I of Arch A_2.

Theoretically speaking, this is true, but from a practical point of view it may be disregarded. An assumption of loading must always correspond to practical exigencies and possibilities of the various combinations of live load crossing a bridge. Now, except for street cars, which in a very abnormal case could fill up both tracks on the bridge on each side of the central span leaving this last one unloaded, the remaining live load, pedestrians, trucks, etc., cannot, for all practical purposes, be distributed according to the assumption suggested.

A dense crowd passing over a bridge is divided approximately into two parts, one-half going and one-half coming, therefore, the most prejudicial hypothesis of loading, in this case, must be that the two side arches are loaded each with one-half the live load. Even then this rather abnormal condition of loading would be of a very short duration, for the crowds keep moving. Hypotheses of loading are circumstantial.

A scientific discussion is very apt to bring forth different points of view; but, when these are candid, as Mr. Godfrey's are, the conflict of opinions is most useful to the reader, who then is able to weigh the arguments on both sides. It is true that there are many causes for uncertainty regarding the relation between the assumptions made in the analysis of an arch and the actual conditions, but it must be said that they have been strongly exaggerated by Mr. Godfrey in his discussion.

The limits of variation between the behavior of the characteristics of the various kinds of concrete, in general, are not so far apart, and the attention

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1145.
† *Loc. cit.*, p. 1155.
‡ *Loc. cit.*, p. 1157.

of Mr. Godfrey is called to the fact that an engineer's assumptions always are, or should be, based on the most unfavorable case. In addition, the writer reduced such assumptions by approximately one-quarter. This seems conservative enough.

According to Mr. Godfrey, Professor G. Lanza has found that "two identical beams showed results almost 100% apart", but Professor Lanza does not state whether this enormous difference was verified below or above the usual limits for the resistance of concrete. Concrete made in laboratories is not, in general, the same as that obtained in fairly well organized construction, where modern mechanical means have reduced the influence of the human factor to a minimum, and where a satisfactory uniformity for the concrete is an accomplished fact.

The hypothesis added by Mr. Godfrey concerning the annealed glass arch, gives a fair idea of his frame of mind on this subject—he is striving for perfection. Now, why should arch design be perfect, while no other structural design is perfect? Every engineer knows that he cannot analyze a structure perfectly; all methods of analysis are mathematically correct, under the assumptions, but their application to this or that structure is always approximate (safe within certain limits). This is due to the fact that some physical phenomena in the structure, thus far, have been found to be of such a difficult and complex nature as to make them uncontrollable by analysis.

If sophism is allowed to invade engineering practice, the whole profession will be unjustly discouraged. The writer is unable to accept the contention that a reinforced concrete arch cannot be considered as an elastic arch. After the results obtained from the tests of the Austrian Committee,* which have demonstrated beyond a doubt that arches made either with a natural or artificial stone fall well within the limits of elasticity, it does not seem necessary to recall any other argument to oppose this contention of Mr. Godfrey.

Concerning the abutments, it is not necessary to add "wasteful mass" to make them practically unyielding. It is for the designer to investigate whether it is more economical to add concrete to the abutment to render it unyielding, or to analyze an arch on the assumption that the abutment yields.

Mr. Godfrey is right when he states that the fixedness of the supports of an arch is seldom verified; but the writer thinks that for practical purposes many abutments may be assumed as unyielding. As previously shown, there is a way of taking care of a rotation of the abutment, when necessary.

Mr. Godfrey advocates the analysis of an arch on the assumption that it is made up of voussoirs, instead of being monolithic as it is. This procedure would put engineering back more than half a century, and would eliminate only a few of the misgivings mentioned by Mr. Godfrey, with the additional inconvenience of making arches unjustifiably heavy. If the quantity of materials used in abutments is "wasteful", certainly this practice would be extravagant.

* Bericht des Gewölbe Ausschusses, *Zeitschrift* des Osterreichischen Ingenieur und Architekten Vereines, Wien, 1895.

There are also scientific reasons for not following this theory. By analyzing an arch system under the old theory of the single arch, it would be difficult to investigate the effects on an arch produced by the loading of another arch of the same system. Mr. Godfrey cannot deny that an arch, under the action of loading, deforms. If so, the arch is an elastic solid and, as such, its analysis may be carried out by the theory of elasticity.

Mr. Harder's argument seems to be that since certain engineers use a given method, all other engineers should follow the same procedure. He states that, since the ellipse of elasticity is not studied in certain schools, it should not be used by engineers; he also adds: "However, the pendulum may swing in the other direction, the ellipse of elasticity may be dusted off, and again the polars will rotate about the anti-poles."

The matter of a scientific principle has no connection with fashion. Whether or not the principle is a good one is of most moment, rather than whether or not certain schools are using it. In the writer's opinion, the ellipse of elasticity will always remain at its own scientific height, helping engineers in the rapid solution of complicated problems for which the application of the analytical methods would be either unpractical or extremely difficult.

Mr. Harder criticizes this theory because it does not take shear into consideration, citing the opinion* of George F. Swain, Past-President, Am. Soc. C. E. The writer recommends that Mr. Harder be not misled by the impropriety of the style which Professor Swain chose to adopt in those remarks, but that he read Paragraphs 21,† 21 (*a*),‡ and 21 (*b*),§ of the Appendix, which cover this matter completely. From these demonstrations it is hard to understand how such an objection can have any weight whatsoever.

There is no need of any classification of structures "in which the influence of the web members on the deflection of the structure is negligible". This method is quite general, and depends on the skill of the designer to take or omit the consideration of shear in his particular problem. Science cannot be reduced to recipes and prescriptions. That there may be cases in which the deflection due to web members may reach 41%, as found by Ritter, nobody will deny; but these are very rare, and any designer, to be worthy of the name, must be able to distinguish.

Mr. Harder has some criticism to make on this method, "because of inaccuracies and awkwardness of some of the construction (for example, the small force polygon with a pole at P_5‖). Any error in this small force polygon may mean a large error in the influence line for the horizontal thrust, p_5." It should be noted that the force polygon, P_5, may be multiplied by a constant to make it as large as the force polygon, P_1, or larger. In connection with

* "On a New Principle in the Theory of Structures," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 702 *et seq.*

† *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1172.

‡ *Loc. cit.*, p. 1173.

§ *Loc. cit.*, p. 1174.

‖ In Mr. Harder's discussion (*Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1194), this is incorrectly given as p_5.

Table 12 the writer has shown a simplification whereby this polygon need not be drawn.

Concerning the use of graphical methods of analysis, there is some ill-founded mistrust among those who believe that analytical methods are far more exact, in any case. There are causes of error in analytical, as there are in graphical, computations. The mathematical concessions that a designer is compelled to make in his analytical design, in order to render it practical, are causes of errors. Although his arithmetical computations are correct, they represent results only approximate, on account of simplifications adopted at the beginning of his analysis, as well as during the working out of his formulas.

Graphical methods, if well adapted to the problem in point, do not need so many approximations, and they are quick, plain, and uniform; and errors depending on bad draftsmanship can be easily detected and corrected. That the graphics are sufficiently exact is shown by the fact that the results found graphically by the writer, have been checked by laboratory results found by Professor Beggs, with a close approximation that analytical methods of design would find rather difficult to attain.

The writer never had the idea of presenting a complete application of this theory to a multiple-arch system. What he had in mind was whether, in dealing with a multiple-arch system, the consideration of only three arches of the system was permissible. It is true as Mr. Harder notes that in each multiple-arch system there are two end arches and that the stresses in these arches will be different from those of the central arch. The writer assumed that every engineer would take this for granted.

Professor Cross' discussion is marked by many incorrect statements and faulty applications of mathematical principles. The writer will point out only a few errors, each of which is vital to some conclusion which Professor Cross has derived from it.

The name of this theory was given to it at a time when the writer was a mere infant. The first scientist (Culman) who announced this theory called it "the ellipse of elasticity" and after him scores of other eminent men called it by the same name, so that the writer disclaims all responsibility for this name, although he finds it quite justifiable.

Professor Cross thinks that Professor Guidi's assumption, as stated in the paper, does not necessarily involve fixed-endedness for the flanking arches. To this there is but one reply, and this is in Professor Guidi's own words.*

The contention that the theory of the ellipse of elasticity does not take the shearing stress into consideration, and that, therefore, it is not suitable for the calculation of web members of an elastic system, has already been answered. It is interesting to note that other discussors, including Professor Mylrea, having read the same paper, have understood clearly how this theory takes into consideration the web members, if this is desired.

As previously noted Paragraph 21 of the Appendix covers this point and shows clearly how the shearing stress is taken care of in the case of a

* "Teoria dei Ponti," Camillo Guidi, Fourth Edition, p. 473.

solid web. Furthermore, from Paragraphs 21 (*a*) and 21 (*b*) of the Appendix, Professor Cross may see how to take care of the web members of a latticed arch.

Therefore, when he says "This is not clear; indeed the discussion of trusses in this paper seems entirely irrelevant", apparently he has failed to see that the elastic weights are to be applied to the center of gravity of each voussoir, in the case of solid ribs, and of latticed ribs with parallel flanges; and to be applied instead at the poles, in the case when the flanges of a latticed member are not parallel, as shown in Fig. 16 (*d*).* If the designer intends to take shearing stress into consideration, it is only necessary to compute the major axes of the elemental ellipses of elasticity of a system by the formula,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{E}{S} \times \rho^2}$$

instead of by

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

in the case of solid rib; by the formula,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \times s^3}{2\,A' \,.\, f}}$$

instead of

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

in the case of a type of latticed girder with parallel flanges; by the formula,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \,.\, s^3}{2\,A' \,.\, f} + \frac{A \,.\, h^3}{2\,A'' \,.\, f}}$$

instead of

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

in the case of another type of latticed girder with parallel flanges; and, finally, in the case of girders with non-parallel flanges, the designer should follow what is said in Paragraph 21 (*b*). This may "seem irrelevant" to Professor Cross, but most engineers would not agree with him.

In most engineering problems, certainly, it is not necessary to take into consideration the deformation of web members. This is merely a general statement that holds good except in those rare cases in which, either on account of the importance of the elastic system, or on account of the nature of the work that this system is assumed to perform, the designer recognizes the necessity of considering the web members. These are not every-day cases, however, and it is the judicious discrimination of problems that

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1174.

determines a designer's standing. Everybody who is familiar with designing knows that there are elastic systems or members of elastic systems that undergo large shearing stresses; but these are exceptions and not a rule.

Apart from Professor Cross' opening remarks, already treated, his discussion, on the whole, can be considered as offering a set of formulas which are "perfectly general" in their application and which, in the opinion of Professor Cross, could be used in place of the method shown by the writer.

On the contrary, it may be demonstrated that:

First.—Assuming the correctness of the formulas, their actual application would require far more labor than the writer's method.

Second.—The formulas themselves are wrong.

Professor Cross starts by assuming that I_a, J_a, and Z_a are already known to the designer by the time he has finished the first step of his work, that is, the analysis of the arch and pier separately.

From Fig. 20 * (whether it be a latticed arch or a solid rib arch is immaterial to this discussion) it is seen that after such an analysis the designer knows the value of I_a about the vertical diameter, y, of the ellipse; that of the moment of inertia about x conjugate with y; and $Z_{yx} = 0$. Now x is never horizontal, except in the case of a symmetrical arch, and Professor Cross' research is for a "perfectly general" application of his formulas. Therefore, the designer will have to compute J_a about the horizontal, x', Fig. 74, which is by no means an easy task. Similar remarks apply in regard to the value of Z_a.

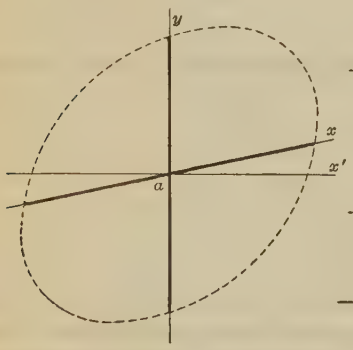

FIG. 74.

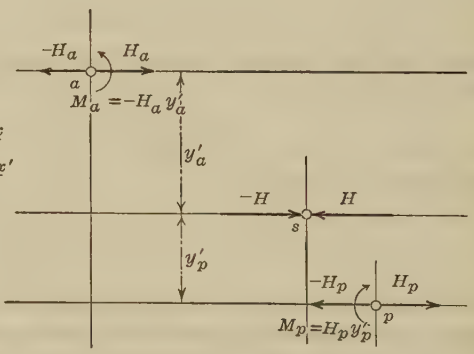

FIG. 75.

By exactly the same line of reasoning it can be shown that the designer will have to compute I_s, J_s, and Z_p, before using Professor Cross' formulas, since the ellipse, s, of the Joint B in a "perfectly general" case, is determined by finding the position of two conjugate diameters, which (except for a symmetrical structure) are never either horizontal or vertical. Only for the pier would the values required by the formulas be already at hand. Thus, the application of these formulas, assuming their correctness, would entail by far a greater amount of labor than the method shown by the writer.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1177.

Concerning the mathematics involved in deriving these formulas: In Fig. 24,* Professor Cross assumes that a is the elastic center for the arch; that p is the elastic center for the pier; and that s is the elastic center for the system (arch and pier). Point s is not the elastic center for the system, but it is the elastic center for Joint B, between the arch and the pier, which is quite a different thing.

In Fig. 25 (a)† Professor Cross shows a certain condition of equilibrium. He applies an horizontal force, $H = 1$, at Point s and two horizontal reactions one at a and one at p with a couple, V, stating that his system is in equilibrium. If, as he states, $H_a + H_p = 1$, it is difficult to understand what couple will balance the couple, V. The presence of the couple, V, means a rotation of the system, and there is no rotation; therefore, equilibrium requires a couple, $-V$, which is missing.

Having assumed that $H_a + H_p = 1$, he implicitly states that the action of the force, $H = 1$, at s, has been balanced; it is therefore logical to assume that the couple, V, does not exist, unless there is a claim to perpetual motion.

In Fig. 75, is shown how the equilibrium of the system should be considered. Applying at s one horizontal, force, H, a force, $-H$, would counteract it and the system would be in equilibrium. Now by dividing the force, H, into two parts, inversely proportional to the distances, y_a' and y_p', they may be imagined as properly transferred, one at a and one at p; in order not to alter the equilibrium, imagine the forces, $-H_a$ and $-H_p$ applied at a and p. Thus, the equilibrium has not been altered.

This transferring and addition of forces, however, has created two couples, one, $M_a = -H_a y_a'$, about a and one, $M_p = H_p y_p'$, about p, which, obviously, are equal and counterbalance each other. The action, therefore, of the force, H, is counteracted by H_a, H_p, M_a, and M_p. Unfortunately, the formulas given by Professor Cross have been derived on the condition of equilibrium as given in his Fig. 25 (a), and, therefore, they cannot be correct.

Furthermore, Professor Cross is wrong when he states: "Since the expression for W_s when I_p or $J_p = 0$, is indeterminate, it is permissible to write" certain equations. The expression for W_s is not indeterminate, when I_p or $J_p = 0$; but it is infinite, which is a very different matter. Therefore, the equation written by him, on account of his wrongly claimed "indetermination" of the expression for W_s, can have no weight.

What is particularly worthy of notice, however, is his assumption that $I_p = 0$. Evidently by that assumption he has quite forgotten that one of the most important points of this research is just the vertical displacements of the supports of an arch, no matter whether this displacement is induced by the shortening of the pier, in general small, or by the compression of the soil. By assuming that $I_p = 0$, therefore, he defeats the very purpose of the method as presented by the writer.

Since all these points have been demonstrated to be utterly incorrect, this formula, like the preceding ones, is to be disregarded. By using these for-

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1199.

† *Loc. cit.*, p. 1200.

mulas, Professor Cross finds data for the pier and fixed-ended arch, which are not very dissimilar from the corresponding data found by the writer. That incorrect methods and formulas should give approximately correct results is simply fortuitous.

Professor Cross then gives a questionable application of the principle of reciprocity to an elastic system. Fig. 27 (*a*)* shows an arch for which he proposes to find the vertical deflection of Point *b* (the key of the arch). In order to find this vertical deflection, he applies a unit horizontal force at Point *a* (elastic center of the arch) and states that the "vertical deflection of $b = i_h \cdot J_a$, where i_h is the influence ordinate at *b* for the horizontal thrust on the fixed arch." This, to say the least, is staggering.

Every engineer knows that, by the principle of reciprocity, the vertical deflection of *b* is found by applying a vertical unit force at *b*, and by computing the horizontal displacement of Point *a*; this horizontal displacement of *a* will be the vertical displacement of *b* when the horizontal force, 1, is applied to *a*. Now, the horizontal displacement of *a*, due to a vertical force, 1, acting on *b*, is given, according to the theory of the ellipse of elasticity, by the force multiplied by the product of inertia of the elastic weights of the arch on the left of *b* (if it is assumed that the left end of the arch is fixed, and the right end is free, and that the center, *a*, is connected rigidly with the free end of the arch); that is, by $1 \cdot \lambda_2 \cdot n \cdot \eta_2$. (See Paragraphs 17, 33, and 37 of the Appendix,† and the top of page 1146 of the paper.‡)

It is this value which, by the principle of reciprocity, represents the vertical displacement of *b* when the horizontal force, 1, acts on *a*. Clearly it is an altogether different value from the one given by Professor Cross. What he did was to imagine that he had found the value of the horizontal displacement of Point *a*, under the action of a unit horizontal force, and to imagine that, by so doing, he had also found the vertical displacement of *b*.

There are two very grave errors in his method. The first is the wrong application of the well-known principle of reciprocity, as shown; the second is that the expression, $i_h \cdot J_a$, as given by Professor Cross, and supposed by him to be the vertical displacement of Point *b*, is nothing but the horizontal displacement of the elastic center, *a*, under the action of a horizontal force, i_h. (See Paragraphs 25, 26 and 41 of the Appendix.§) As a matter of fact the horizontal displacement of the elastic center, *a*, under the action of the horizontal force, 1, is $1 \times J_a$.

Whatever the explanation, the fact remains that his value is perfectly foreign to the case in point. This is not a case of bringing *b* to its former position, which return could be obtained by applying at *a* a horizontal force, *H* (i_h of Professor Cross), and, consequently, causing a horizontal displacement of $a = H \cdot \lambda_2 \cdot v \cdot n$ ($i_h \cdot J_a$ of Professor Cross). This is a case of finding the vertical displacement of *b* when a horizontal force, 1, acts on *a*, no matter how much it is going to be displaced. The difference between these

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1204.
† *Loc. cit.*, pp. 1171, 1178–1179.
‡ *Transactions*, Am. Soc. C. E., Vol. 88 (1925).
§ *Loc. cit.*, pp. 1175–1176, 1179.

two cases is really quite clear, the "obfuscation" being purely in Professor Cross' mind.

There is a final mathematical question treated by Professor Cross which certainly needs explanation, when he states*: "Also, a unit moment at a will produce a vertical deflection at b equal to $i_m\ W_a\ e_y'$, in which, i_m is the influence ordinate at b for the moment at the neutral point of the fixed arch."

This is possibly even more questionable than the point just mentioned. It is well-known that by the principle of reciprocity, the vertical displacement of Point b, caused by moment = 1, acting on the elastic center, a, is given by the rotation of the center, a, caused by the vertical force, 1, applied at b. Now, applying a vertical force, 1, at the point, b, gives a rotation of the center, a, and this rotation, by the theory of ellipse of elasticity, is given by $1\ \Sigma\ W\ \eta_0$, in which, η_0 is the ordinate moment of the elastic weights of the arch from the left up to Point b (if it is assumed that the arch is fixed at the left, and free at the right, as previously).

This value represents the vertical displacement of b when a unit moment is applied at a. Thus, it is seen that the correct value here given differs from that of Professor Cross, by a certain distance, e_y,† which, however, has been introduced with no apparent justification. Subsequently, Professor Cross finds that the value of this quantity, $e_y = 0.10$, that is, that the value of the vertical displacement, b, as found by him, is only one-tenth of what it should be.

It is with such formulas that he undertakes the "Study of Crown Thrust".‡ No other comment is necessary, except to say that evidently Table 9,§ with all Professor Cross' assumptions, needs a radical revision.

Thus, it seems that Professor Cross, with very little consistency, has treated with contempt the theory of the ellipse of elasticity, while on the other hand he has used and misused it throughout his camouflaged algebraic method; but that he should commit errors in other theories which apparently are accepted by him, seems unpardonable.

Professor Cross states‖, finally, "the writer believes that a correct and practically complete analysis of a multiple-arch system is quite feasible," etc. This is merely a statement, and in mathematics a statement has no value if not supported by demonstration. It would have been quite proper for him to have presented a practical illustration which would have been entitled to consideration.

As to the statement¶ in the beginning of his discussion that, in America, "college graduates know little about properties of poles, polars, and anti-poles, and are, perhaps, not the worse for the deficiency", if college graduates know so very little about these properties that they cannot grasp the meaning of

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1203.

† *Loc. cit.*, p. 1204. In Fig. 27 (*b*) of Professor Cross' discussion, "c_y" should read "e_y." Text is correct.

‡ *Loc. cit.*, p. 1203.

§ *Loc. cit.*, p. 1205.

‖ *Loc. cit.*, p. 1206.

¶ *Loc. cit.*, p. 1197.

principles founded on them, it is scarcely worth while wasting their time in studying the subject at all.

Mr. Hammill, who has used this method for several years, has found it to be more accurate than an analytical one. His conclusion could not be otherwise. Those in favor of exclusively analytical methods apparently forget the approximations they have to make in solving their equations. Furthermore, when a graphical method can be checked, at any moment, by simple slide-rule computations, all criticisms against the method merely show a lack of familiarity in more advanced theories.

A well-known professor once said, in substance, that graphical methods were unreliable for even the thickness of a line may mean a difference of value. The writer is not sure whether this professor had in mind the lines drawn by carpenters in their daily work, or lines drawn by any average designer. In the first case he was right; in the second, apparently, he was not quite familiar with graphics.

Professor Beggs has supplied this paper with a very interesting complementary work. The writer will not enter this field for it is so clearly explained by its author. His method for testing models is ingenious and his results as regards the triple arch treated by the writer are very gratifying, for they show that the analysis of this system has been correctly applied. These results seem to answer the criticism that graphical methods are inaccurate.

The difference recorded in the diagrams between the theoretical values of the moments and the observed values of the moments on the template, in the writer's opinion, are due to unavoidable imperfections of the small model, and to some slight friction which develops in its deformation during the test.

Mr. Hudson is right when he calls "unusual" the type of structure taken for example by the writer; but, evidently, he has not thought that this structure was purposely taken as an example, for its make-up is such that the difference in results obtained from the various combinations of the arches would be more apparent. There was no intent of preparing plans for a system of arches to be built for every-day use. The idea was to present a method to be applied to a system of arches. In many books the diagrams explaining theories and methods of design are not drawn for actual building, but to bring home the truth in the best way.

Mr. Cohen, like Mr. Hudson, remarks concerning the geometry of the structure. It is justly remarked by Mr. Cohen that in the paper no attempt is made to show the applicability of the analysis presented to economic conditions. The writer thinks that any such attempt would be fruitless. Economical conditions, it may be said, vary almost in each and every instance, and any attempt at standardization would not be serious.

The correction suggested by Professor Mylrea concerning rib-shortening is proper. This error was an oversight on the part of the writer.

The writer fully agrees with Professor Schachenmeier concerning the convenience of having a combination of a graphical and analytical method.

The method of the ellipse of elasticity, although presented in a purely graphical way, can nevertheless be checked at each step by simple computations, as has been shown.

Most of Mr. Whitney's remarks and objections find their answer in the results shown in the first part of this closing discussion. He seems to have stopped at the names and titles of books dealing with this theory; for instance, he cites the name of M. Sejourné as one having dealt with this theory as applied to arch design. Had he gone further, he would have found that Paul Sejourné acknowledges that the method he explains has been taken from Guidi's work.

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Paper No. 1636

GRIT CHAMBER PRACTICE

A SYMPOSIUM*

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* Presented at the meeting of the Sanitary Engineering Division, Montreal, Que., Canada, October 15, 1925.

GRIT CHAMBERS FOR SEWAGE TREATMENT WORKS

By George B. Gascoigne,* M. Am. Soc. C. E.

Synopsis

This paper is concerned with emphasizing one phase of grit-chamber design, namely, the possibility of obtaining deposited material sufficiently free from organic matter to be of use for non-offensive filling. Experiments conducted at Cleveland, Ohio, with this end in view, are quoted; also two major installations are described wherein the desired object has been attained.

Reasons for Grit Chambers

Inorganic substances which find their way into sewerage systems, and particularly into those of the combined type, must generally be removed from the sewage if economic and satisfactory treatment is desired. Grit removal therefore becomes a preliminary to other and more elaborate stages in the general process and it is applicable to practically all methods of sewage treatment. In the early life of some sewerage works, the grit-chamber treatment has been omitted, especially when the separate system of sewers is used. Noteworthy for such omission are the installations at Baltimore, Md., and Columbus, Ohio. However, recent and critical studies of separate sewerage systems are demonstrating more and more that grit-chamber treatment is, if nothing else, a precautionary feature that justifies its inclusion in the process. Grit chambers also serve as safeguards to the operation of pumping machinery, and permit moving equipment, such as valves and gates, to function properly. They prevent deposits in siphons and in other parts of sewerage works where removal would be laborious and expensive, and they further permit of more efficient disinfection of either a crude or partly treated sewage. However, probably more important than these is the fact that grit chambers provide, at a relatively small cost, a satisfactory means for removing the inorganic substances that enter sewerage systems.

What Is Grit?

There is probably no misconception of what grit actually is. The standard definition of the American Public Health Association is that "Grit is the heavy mineral matter deposited from sewage". However, the material actually obtained as grit includes not only the inorganic substances of sewage—the detritus of English terminology—such as sand and cinders, but also more or less inert organic matter, such as coffee grounds, fruit seeds, and, of late years, mash and similar substances. Tar products are also often present. In addition to these materials, which may be regarded as proper constituents, it has been general experience that grit usually contains more or less organic matter which belongs in the sedimentation tank. The presence of such material

* Cons. San. Engr., Cleveland, Ohio.

affects and complicates the ultimate disposal of the grit. This paper is largely concerned with efforts to deal with the problem of reducing this content to acceptable proportions.

Grit Chambers Prior to 1910

Grit chambers were a general part of early European treatment works. As Dr. Dunbar* points out, they probably developed in consequence of deposits occurring at the gratings used to protect sewage pumps.

In the development of treatment processes, it is significant that the importance assigned to a preliminary removal of inert solids varied with the type of treatment. Broad irrigation projects found little advantage in grit removal. Conversely, the designers of chemical treatment works were likely to include grit chambers as an essential feature of the plant. With septic tanks and in the early sedimentation tanks, practice was far from uniform. It is quite evident that lack of basic data prevented determination of how grit removal was related to the general problem of sewage treatment.

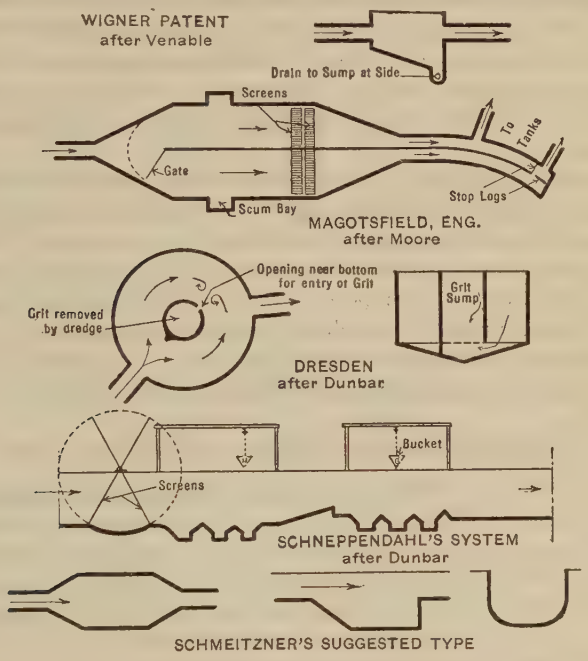


Fig. 1.—Some Early Forms of Grit Chambers.

It is true, however, that generalization is prone to ignore local factors which may have led to designs for which reasons do not now appear. Also, it is noteworthy that the omission of grit chambers at Columbus was based on elaborate experiments in large working units.

Despite the absence of any marked trend in grit-chamber design during the early years, it is interesting to observe the variant views of engineers as

* "Principles of Sewage Treatment," translated by H. T. Calvert, 1908.

reflected by the installations in German and English plants of the middle Nineties. Some of the forms are shown on Fig. 1. American practice appears to have ignored the problem or to have regarded grit chambers as a minor feature warranting little expense for construction.

It was natural that the greater demands of subsidence and of the treatment of the strictly organic content of sewage should have deferred experimental study of grit-chamber characteristics. Winslow and Phelps at the Experiment Station of the Massachusetts Institute of Technology during 1904-06, made determinations of grit removal. Brown, at Rochester, N. Y., in 1909, conducted experiments on critical velocities. Reference has been made to the extensive experiments of Johnson at Columbus in 1905. In Germany, both Dunbar and Schmitzner arrived at conclusions regarding a proper velocity of flow, slopes, and form of chamber. The Royal Sewerage Commission of England suggested desirable detention.

The mechanical removal of grit did receive considerable attention, and it is noteworthy that present-day practice marks no decided advance in this respect over the installations of the early works. Examples of these are found in the endless chained buckets at Manchester, and the traveling crane at Birmingham, England, and in the suction device tried at Düsseldorf, Germany. In the smaller plants, as at present, the chief reliance was on hand cleaning.

In summarizing practice prior to 1910 one finds isolated investigation and experiment, but generally there appears to have been no consistent or continuous appreciation of the proper function of grit chambers. In this it is probable that the often putrescible character of the deposited material had much weight. Indeed, in his report to the late R. Winthrop Pratt, M. Am. Soc. C. E., in 1908, Kimberly* concluded from his examination of thirty-eight plants in Ohio that grit chambers were worthless as preparation to the septic process, and based his findings on the offensive nature of the grit.

Experimental Grit Chambers at Cleveland, Ohio

Treating the sewage of Cleveland is one of the major sanitation problems of the United States. When the program of improvement was initiated, the magnitude of the project was deemed to justify considerable effort to establish with some certainty the accomplishments and costs of various methods of sewage treatment, in their local application. The City of Cleveland undertook extensive and, as it later proved, creditable work in a study of sewage treatment methods. Parenthetically, it may be noted that this work was done at relatively small cost in comparison with both the cost of the sewerage program and the amounts expended elsewhere for similar purposes. In 1912 the structures for the Main Testing Station were designed, constructed, and made ready for continuous operation. These investigations to determine the most satisfactory and economical method for treating the sewage of Cleveland were begun in January, 1913, and continued for an operating period of more than one year. The results of this investigational work were compiled,

* "Water and Sewage Purification in Ohio," State Board of Health, 1908.

tabulated, and published by Messrs. Hoffmann, Pratt, and Hommon, under date of December 15, 1924.*

Although the work at the Testing Station comprised an investigation of grit chambers, hand-cleaned bar gratings, and coarse screens, sedimentation in various types of tanks, roughing filters, trickling filters, disinfection, and sludge treatment, particular stress was placed on developing a workable grit chamber and especially one that would produce a grit suitable for disposal as fill for making land in residential districts. To the best of the writer's knowledge these particular tests were the first in which stress was placed on the necessity of obtaining a suitable and inoffensive grit, and credit for the idea is given here to whom it belongs, the late Mr. Pratt.

The original grit chambers (Fig. 2), through which the sewage first entered the Testing Station, were to operate at velocities of about 10 ft. per min. This arrangement was in accordance with ideas then obtaining, but the chamber was constructed so as to allow for any alterations that might be desired. Changes were made from time to time as shown on Fig. 2. These changes were directed mainly to increasing the depositing velocity thus affecting the quality of the material deposited.

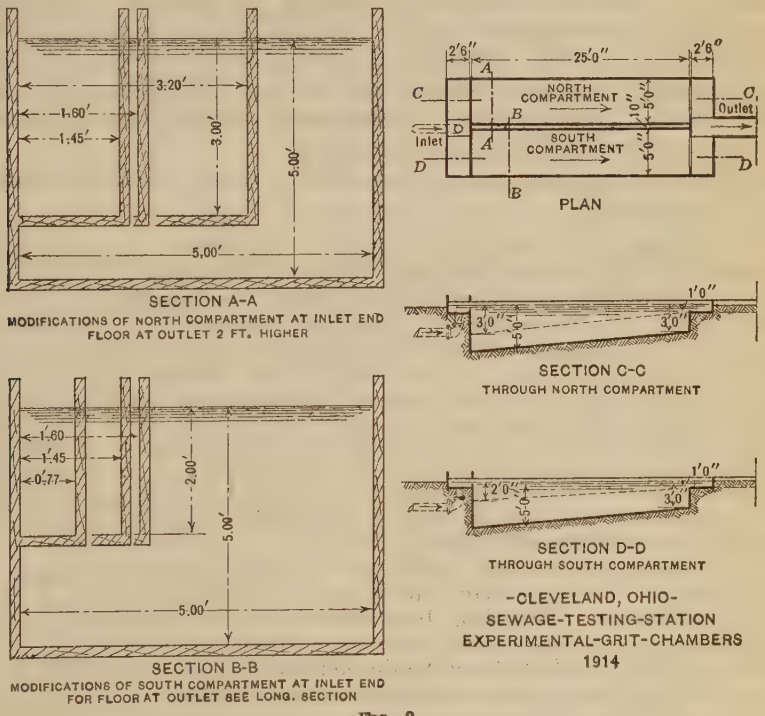


FIG. 2.

The grit-chamber studies included runs of periods varying from 4 hours for a storm flow to 167 hours for a dry-weather flow. Studies were also made

* Report on Tests, Sewage Testing Station, 1914.

of the rate of deposition, particularly during periods of storm. The conclusions reached from the experiments may be summarized as follows:

1.—That grit was always present in a combined system of sewers.
2.—That grit could be effectively removed as inoffensive material, suitable for filling purposes in residential sections.
3.—That to be inoffensive, the grit should not contain on the average, more than 15% of volatile matter.
4.—That with an average detention period of 50 sec. during dry-weather flows, approximately 0.6 cu. yd. of grit per 1 000 000 gal. would be deposited, and that this would be increased to 0.7 cu. yd. during storm flows. This amounted to approximately 0.6 ton of dry material per 1 000 000 gal.
5.—That with velocities varying from 30 to 60 ft. per min., grit of proper character, suitable for fill without causing offense, would be obtained.

It is significant that the results obtained from the permanent grit-chamber installations in Cleveland have practically substantiated these conclusions except as to the quantity of grit deposited per unit of dry-weather sewage. The first conclusion regarding the general presence of substantial quantities of grit has not been fully established, but it has not been disproved. As regards the quantity of grit which would be deposited, the permanent installations indicate that the experimental results were too high, at least, in so far as normal flow is concerned. Probably one of the reasons for this is that the sewage intake for the investigations was too near the bottom of a main trunk-line sewer in which velocities of flow were relatively slow and deposits of grit were likely to occur and to accumulate.

The following quotations are of general import and emphasize particular points of the tests:*

"It was evident, in making the various runs, that of the entire amount of grit retained in the chambers, approximately two-thirds was deposited in the first or inlet half of the chamber, therefore in determining the weighted averages for the volatile matter, the three assumed sections of the chambers, inlet, middle and outlet, were given weights of three, two and one, respectively. A grit deposited in the outlet section may contain volatile matter in excess of fifteen per cent., but when the quantity is taken in combination with the total, the whole is not objectionable provided the average is less than fifteen per cent. Certain runs given in Table No. 19 [of the Report on Tests] are selected because it is thought that they represent what the grit chamber actually accomplished. . It was found that when the total deposited exceeded a certain amount, grit was carried beyond the chamber, therefore, all the runs are omitted in which this condition occurred.

"Two methods were used in measuring the volume of grit deposited. Up to July 1st, the quantities were determined by measuring the depth of the grit in the chambers. After July 1st a special box was mounted on the side of the chamber and the grit was measured in this after it had been allowed to drain. It was found that the results obtained by measuring the grit in the box were about 16 per cent. higher than those obtained by measuring the grit in the chambers. The quantities recorded in Table No. 19 [Report on Tests] were determined from measurements of the grit in the chambers.

"From Table No. 19 [Report on Tests] it is evident that with an average detention period of approximately fifty seconds during dry weather flows there is deposited 0.6 of a cubic yard of grit per million gallons of flow, and during

* Report on Tests at Sewage Testing Station, City of Cleveland, 1914.

storm flows, 0.70 of a cubic yard per million gallons of flow. This removal is equivalent to 0.54 and 0.63 tons of dry material, respectively. In general, velocities less than 25 feet per minute do not produce the proper grade of grit and in the 25-ft. chambers velocities of sixty feet usually carried by some material which should have been deposited."

Demonstration Grit Chambers at Cleveland, Ohio

Experiments conducted at the Main Testing Station during the original investigations in 1913 and 1914, indicated that the fine screening process offered some particular advantage at the Westerly site in Cleveland, and that certain fundamental features of the screening process could not be thoroughly studied except with the sewage to be treated. The idea, therefore, of testing a sewage screen of substantial proportion was in line with definite recommendations and provided for determining, by actual tests, the most satisfactory and economical process of partly treating the sewage at the Westerly site. In conjunction with the screening plant, it was necessary to build two full-sized grit chambers (Fig. 3).

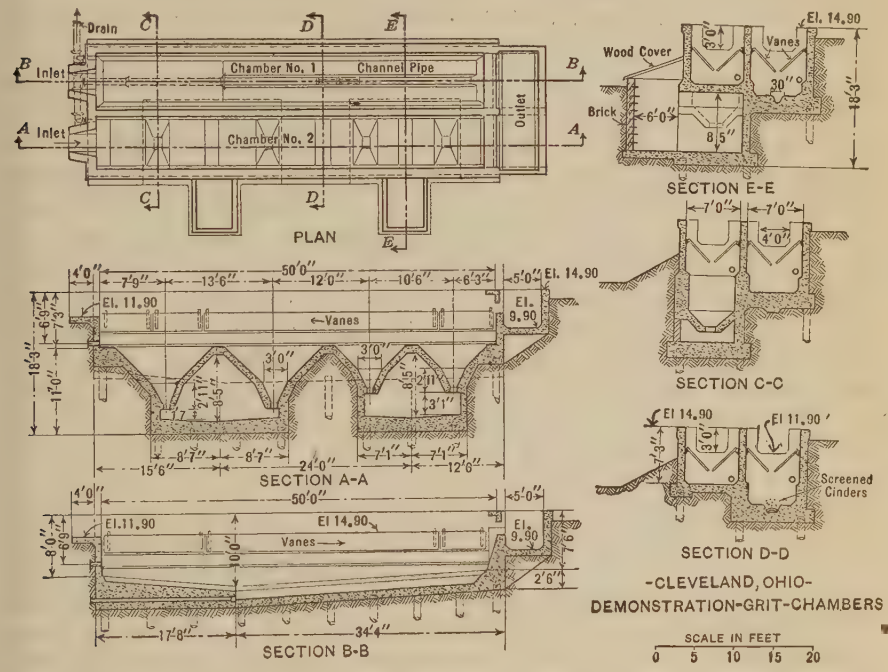


Fig. 3.

At the time of constructing these demonstration grit chambers the Westerly interceptor discharging at this site served a population estimated at 50 000 (1916), and furnished the sewage which was treated by these chambers. On entering this plant, crude sewage first passed through one set of bar gratings, situated in the flow channels at the entrance to the grit chambers. These gratings were to prevent large objects such as boards, pieces of wire or tin, rolls of paper, etc., from reaching and interfering with the opera-

tion of the fine screens, valves, and control gates of the plant. The sewage then passed to either of two grit chambers, one of which was divided into two parts, an upper or flow section and a lower or deposition section. The other chamber had a bottom with four hoppers into which the grit was to deposit and from which it was expected to demonstrate the removal of the deposited grit by ejectors without interruptions to sewage flow. Both chambers, as shown on Fig. 3, were 50 ft. long and designed to give a velocity of 1 ft. to 0.5 ft. per sec., with sewage flows varying from 8 000 000 to 16 000 000 gal. per 24 hours. The length permitted a maximum detention period of 50 sec. and it was estimated that the average deposit of grit would be 0.5 cu. yd. per 1 000 000 gal. of sewage flow.

Operation.—In the operation of the demonstration grit chambers particular attention was given to the satisfactoriness of certain special or unusual features of grit-chamber design. These may be listed as follows: (1) Hoppered *versus* flat-bottomed chambers; (2) control of entrance and depositing velocities; and (3) removal of grit by ejectors.

During the experimental work it was found that grit would not deposit uniformly throughout the entire length of a chamber, and that that deposited in the down-stream portion contained excessive quantities of organic matter. This occurred even when the grit in the outlet end of the chamber was being washed or cleaned supposedly by the sewage which passed over it. Although this cleaning action must occur to a certain extent, it is a fact that relatively large quantities of organic matter become entrapped by the depositing grit, especially just after a cleaned grit chamber is placed in operation or where the effective velocities are relatively low. This action increases the organic content and the likelihood of nuisances in the disposal of such grit, and in practice it is difficult to prevent. The washing action that is supposed to occur with the use and in the operation of a flat-bottomed grit chamber is more theoretical than real, especially when the grit-removal problem is viewed from the standpoint of obtaining a grit that can be disposed of as fill in residential districts. It is this lack of washing which suggested hopper-bottomed grit chambers to the writer, particularly since the washing or releasing action may be brought about by causing sewage with grit to pass over a series of pockets. In this way the grit would be released or deposited when over the pockets and the sewage and grit given a stirring action when passing through the restricted sections or between the pockets. While this idea is fundamental in the use of hoppers or pockets for a grit chamber there are other factors, especially that of simplification of the cleaning problem. However, these several viewpoints brought about the installation of the two types of bottoms in the demonstration plant. Observations of the operation of the alternative types and of qualities of grits collected therefrom verified the reasoning given and led to the acceptance of hopper-bottomed grit chambers for later installations. There was no way, except by watching the deposits in the open outfall sewer leading from the plant, of determining whether or not some grit was passing the grit chamber, which ordinarily should have been deposited. In other words, it was believed in some instances to be safer

to have some of the finer grit carried by or deposited elsewhere in a plant where it must eventually be removed than to deposit in the grit chambers some organic matter which would later cause nuisances.

The writer does not believe there are any marked differences of opinion regarding the theoretical velocities which obtain with a hopper-bottomed grit chamber, but there is material difference as to the practicability of maintaining theoretical velocities in grit chambers of rectangular cross-section. This is true because the effective cross-sections of a rectangular grit chamber change as the deposits increase. The practicability of controlling these velocities by providing a number of units, by baffling, by shifting the floor, or otherwise, is questioned seriously. The writer is, therefore, of the opinion that hopper-bottomed grit chambers lend themselves more readily toward producing in practice the theoretical velocities used in design.

If it is necessary to obtain with grit-chamber treatment certain limiting velocities for a rather definite period of time, particular stress must be placed on making the entire length and width of the chamber effective. To do this each grit chamber of the demonstration plant was provided with a separate set of bar gratings, a procedure which the results demonstrated to be fully justified. Such provision reduces the length of inlet channels and distributes uniformly the sewage as it enters the grit chamber. Although the writer has used two sets of bar gratings primarily for another purpose, the second set aids further in distributing evenly the incoming sewage.

In starting the operation of the demonstration plant, the flat-bottomed grit chamber was divided into two compartments, an upper or flow section and a lower or deposition section. The flow section was formed by attaching to the vertical side walls steel plates or vanes and hanging them so that they converged to a slot in the center. It was contemplated in this construction to maintain the depositing velocity throughout the entire length of the grit chamber and thus to separate the flowing sewage from the deposits in the bottom. To the best of the writer's knowledge, there was never any intention of moving or regulating these vanes in accordance with the fluctuation of the flow, although this might have been possible. Operating results showed rather quickly that grit obtained with these devices had a large organic content. This was due primarily to the fact that the settleable and organic solids of the sewage, entrapped behind the vane and above the grit deposits, settled out into the grit. Consequently, the use of the vanes was discontinued.

Sand ejectors were fastened to the bottoms of the four hoppers and were operated by city water pressure. The material thus ejected was to be deposited at a substantial distance from the chambers. There was also included the idea that the grit chambers might be cleaned when the sewage was flowing through them. A rather determined attempt was made to prove the utility of sand ejectors, especially as their use would have permitted a substantial saving in operating cost. Grit with tarry products and stones that vary in size from 0.5 to 1.5 in. cannot be removed by sand ejectors as installed in the demonstration plant. In other words, if sand ejectors are to be used satisfactorily, the sewage must be fine-screened before entering the grit chamber.

This idea, which appeared quite promising at the beginning of operations, proved useless in the final designs. Eventually, the grit chamber with the four hoppers had the second and fourth hopper sealed with a wooden floor, to form a chamber that resembled in many respects the two hoppered chambers finally used in the designs for the Westerly and Easterly Works. The practicability of using two hoppers was demonstrated in this plant.

Design of Grit Chambers

Quality of Grit.—At the outset in any design, it is advisable to fix on the desired quality of the grit to be deposited. By quality is meant the proportion of organic content that may safely be carried. This determination is largely local, with the ultimate disposal of the material as a prime factor. Where sites are remote and secluded, the limits need not be so rigid as where residences adjoin and where dumping areas are restricted. A low organic limitation may result in the passage of the finer grit to later processes, and there must be included an appraisal of the effect of such material on these modes of treatment. At Cleveland it was judged that an organic content of 15% provided economic balance, but this percentage is not applicable generally. From the desired standard of quality there results velocity limits which can be translated into terms of the cross-section and number of grit-chamber units. This last is also a function of the extent and range of flow. Multiplication of channels is essential to adequate control, but the practical considerations and the construction expense must also be heeded. Also, the fluctuation in any one unit must not be extreme. The length of the chamber fixes the detention period, which under normal conditions may be set at 50 to 60 sec. for average flow.

Quantity Removed.—The quantity of grit for which storage must be provided is to be fixed to meet the maximum storm demand. The cleaning cycle and the number of units held in reserve also affect this. The maximum rate of deposit may well be determined by experiment, or failing that, by the conditions which obtain at sewer outfalls. It is distinctly a variable to be established from considerations of terrain and development, and also in the light of the extent and of the velocities of the tributary sewers. Combined sewers, of course, will yield results different from those which apply to so-called sanitary sewers or to interceptors. The Cleveland experimental maximum of 0.7 cu. yd. per 1 000 000 gal. seems to be entirely too high for interceptors from combined sewers. Where the extent of cleaning equipment must be determined, or the area for dumps, local experience establishes 0.1 cu. yd. per 1 000 000 gal. as a suitable figure.

As regards quantity of grit in Northern latitudes, there is a tendency to lengthen the winter-time intervals between cleanings. Any provision for this must be reflected in the storage space.

Depositing Velocities.—The work at Cleveland has not particularly extended available data concerning velocities of deposit. In the experimental unit the lower velocities—proximate to 10 ft. per min.—were quickly abandoned. Successful operation was obtained in the practical range of 30 to 60 ft. per min. These figures accord with usual practice.

It is further true that both the length of travel (detention period) and the depth of sewage are effective in determining the rate of grit deposit. In the matter of depth there must be eliminated what Frank terms the "lower slow motion zone in which the settling organic sludge particles may be entrapped".* At Cleveland, this factor is apparent at the Testing Station where the tank bottom was raised from a depth of 5 ft. at the inlet end to depths of 3 and 2 ft. (See Fig. 2.) To meet this requirement, the permanent installations were built with channel depths of from 3.2 to 5.55 ft., when carrying a daily flow of 24 000 000 gal. In contradistinction to these channel depths are those of the pits, which range from 6.05 to 8.84 ft. below maximum water line.

Cleaning Devices.—The method of cleaning grit chambers is largely dependent on the size of the installation. For small plants in which duplication of units has been provided it is usually most economical to depend on hand-cleaning. In such cases the chamber is by-passed, the water removed from the grit through the under-drains, and the materials shoveled into dump cars or wheel-barrows and hauled to the spoil area.

In larger installations it is found more economical to use some type of mechanical equipment that not only requires less time and labor but also permits the chambers to be cleaned while still in operation. Among the various mechanical devices used for such purposes may be mentioned (1) eductors; (2) bucket elevators; and (3), clamshell buckets on mounts of various types.

Consideration of the several factors of first cost, operating charges, dependability, and ease of control led to the adoption at Cleveland of a clamshell bucket, mounted on a monorail and controlled from a cab, which moves with the bucket. From an operation standpoint this is particularly convenient as the operator is over his bucket at all times, and can readily guide it in dumping, more particularly when entering the relatively narrow compartments and pits. At both plants the monorail is formed into a loop, which passes over all pits and which is convenient to the industrial track that carries the cars to the dump. The operation of the bucket is electrical.

Features of Design.—A review of the installations at the Easterly and Westerly Plants shows more in the way of equipment than is usually found in the grit-removal portion of sewage treatment works. Such devices as hydraulic sluice-gates, drains for supernatant sewage in dewatering tanks, and under-drains for seepage from grit are in use. These features may not be essential, but they conduce greatly to ease of operation and to the ready control which are attributes of the installations here described. The provision of bar gratings in each unit may affect the labor cost for cleaning, but are regarded as desirable in ensuring even distribution of flow.

Grit Chambers at Easterly Sewage Treatment Works, Cleveland, Ohio

The Easterly Sewage Treatment Works forms one of three major units intended to care for the sewage of Cleveland. It serves the larger part of the city, the district being chiefly urban, but also including suburban develop-

* "Design and Operation of Imhoff Tanks," by Leslie Frank, Frank Fries, and Charles Gilman Hyde, 1914.

ment. The works are on the lake front at East 140th Street. Although designed to include sedimentation, initial construction at this plant was confined to the installation of gratings and of grit chambers, together with disinfection equipment and an outfall sewer. In this form the works have been operated since 1921.

The grit chambers at the Easterly Works are typical of the Cleveland form, evolved as a result of the 1913 experiments and from subsequent studies on large-sized units. They include a dry-weather flow channel and twelve grit-chamber units. The grit chambers are of rectangular cross-section, each 97.5 ft. long from entrance gate to overflow weir. The grit chamber proper is 60 ft. long and 7 ft. wide. The sloping floor leads to two rectangular hoppers or sumps. The depth under maximum flow conditions ranges from 3 ft. at the bar gratings to 8.84 ft. in the up-stream sump. A building, 121 by 145 ft., covers the whole nest of grit chambers and screens, and includes a complete layout of control apparatus. A conventionalized plan and sections are shown on Fig. 4.

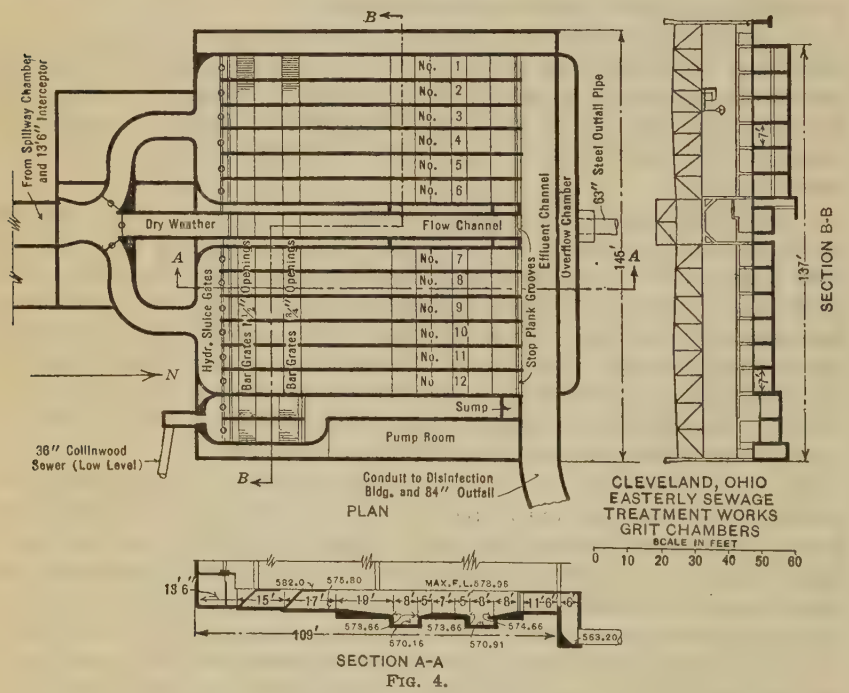


FIG. 4.

The notable features of this installation are the multiplicity of channels, the relatively small cross-section of each unit, the provision of bar gratings in each chamber, and the grit pits or sumps. The arrangement of central channels, the hydraulically-operated gates, and the drains for dewatering the tanks and for draining the pits are also on a scale appropriate to major works. The building is substantial, heated, and well equipped. Cleaning is accomplished by the aid of an electrically-operated grab bucket traveling on a monorail system, loading into industrial cars.

The Easterly Plant was designed to serve a population of 575 000 with an average dry-weather flow of 92 000 000 gal. daily. Each grit-chamber unit is proportioned for a flow of from 12 000 000 to 24 000 000 gal. daily; consequently, if all 12 units were in operation, the maximum flow capacity of these chambers would be 288 000 000 gal. daily.

The designed velocities were from 0.5 ft. to 1 ft. per sec. and the detention period 60 sec., the maximum period being 2 min. when the lower velocity occurred. The grit storage space is 13 cu. yd. per pair of hoppers about equally divided between the up and down-stream parts of the chamber, which means a total provision of 156 cu. yd. in all 12 grit chambers. In addition to the pit capacity, there are available in gross, about 243 cu. yd. of space below the flow line of the outlet channel. Under maximum or storm-flow conditions the available capacity per 1 000 000 gal. would be roughly 1.38 cu. yd., and if the number of chambers effective is reduced 25%, this figure is decreased roughly to 1 cu. yd.

The works as described have been in use since 1921. From the operation records the data in Table 1 have been taken. This information was made available through the courtesy of J. W. Ellms, M. Am. Soc. C. E., under whose general direction the sewage treatment works of Cleveland are operated.

In considering the records of Table 1, it should be appreciated that the limiting days are end dates and represent the terminations of cleaning periods which averaged perhaps three days, but which in many cases lasted much longer. This fact will account for some of the variations. Others may be ascribed to storms of unusual intensity and duration, as those which occurred during the summer of 1924.

The frequency of cleaning is of course a function of the storm flow and of storage capacity. During the 2½ years of this record, the range was from once in 5 days to a winter-time suspension of cleaning for 39 days. The average period between cleanings was about 16 days. The average yardage removed at each cleaning represented about two-thirds of the total storage capacity of the pits, and the maximum exceeded the total pit capacity.

The records as kept show little regarding what may be termed the intensity of deposit. The periods in July, September, and October, which show maximum rates of deposit, are those which cover certain heavy storms of 1924. It might indeed be expected that entrainment of grit would exhibit a variation somewhat similar to that shown by factors representing storm intensities and time of duration. It would be necessary, however, to compensate for other items, including antecedent periods of drought or moderate rains. Table 1 does not enable the determination of such maxima, but the average values of 0.26, 0.23, and 0.27 cu. yd. per 1 000 000 gal., as noted for the peaks, give rise to the suspicion that the Cleveland Testing Station was not much out of line in fixing 0.7 cu. yd. per 1 000 000 gal. as a value appropriate to storm flows for a 50-sec. detention. However, in comparing the Testing Station data with those of operation, there is apparent quite a wide discrepancy in the dry-weather deposits. At the Testing Station it was found during dry-weather intervals that 0.58 cu. yd. of grit was obtained for each 1 000 000 gal.

TABLE 1.—Record of Grit Removal, Easterly Plant, Cleveland, Ohio, Sewage Treatment Works.

Period of run.	Days.	Grit removed, in cubic yards.	Sewage flow, in millions of gallons.	Grit, in cubic yards per 1 000 000 gal. of sewage.	Analysis.		
					Date.	Volatile and combustible percentage.	
						Up stream.	Down stream.
Feb. 6-Mar. 3, 1923..	26	65	2 210.1	0.029	.....	.....	.....
Mar. 4-Mar. 13, 1923..	10	51	876.6	0.058	.....	.....	.....
Mar. 14-Apr. 3, 1923..	21	67	1 908.5	0.035	.....	.....	.....
Apr. 4-Apr. 10, 1923..	7	82.5	688.8	0.120	.....	.....	.....
Apr. 11-Apr. 25, 1923..	15	70	1 309.6	0.053	.....	.....	.....
Apr. 26-May 10, 1923..	15	79	1 296.0	0.061	.....	.....	.....
May 11-May 18, 1923..	8	29.3	801.3	0.037	.....	.....	.....
May 19-May 23, 1923..	5	43	420.7	0.102	.....	.....	.....
May 24-June 2, 1923..	10	83	817.8	0.102	.....	.....	.....
June 3-June 12, 1923..	10	93	860.8	0.108	.....	.....	.....
June 13-June 26, 1923..	14	92	1 205.8	0.076	.....	.....	.....
June 27-July 10, 1923..	14	82	1 199.8	0.068	.....	.....	.....
July 11-July 24, 1923..	14	82.5	1 260.8	0.065	.....	.....	.....
July 25-Aug. 6, 1923.	13	97.5	1 194.5	0.082	.....	.....	.....
Aug 7-Aug. 17, 1923..	11	101	835.4	0.121	.....	.....	.....
Aug. 18-Sept. 1, 1923..	15	151.5	1 176.5	0.129	.....	.....	.....
Sept. 2-Sept. 15, 1923..	14	149.5	1 152.9	0.130	.....	.....	.....
Sept. 16-Sept. 26, 1923..	11	116.5	993.9	0.117	.....	.....	.....
Sept. 27-Oct. 9, 1923..	13	110	1 160.4	0.095	.....	.....	.....
Oct. 10-Oct. 23, 1933..	14	93.5	1 246.6	0.075	.....	.....	.....
Oct. 24-Nov. 7, 1923..	15	136	1 595.5	0.085	.....	.....	.....
Nov. 8-Nov. 20, 1923..	13	96.5	1 290.8	0.075	.....	.....	.....
Nov. 21-Dec. 4, 1923..	14	81	1 484.8	0.054	.....	.....	.....
Dec. 5-Jan. 10, 1924..	37	117.5	3 536.8	0.033	.....	.....	.....
Jan. 11-Jan. 28, 1924..	18	108	1 708.9	0.063	.....	.....	.....
Jan. 29-Feb. 10, 1924..	12	85	1 281.8	0.066	.....	.....	.....
Feb. 11-Feb. 26, 1924..	16	28	1 437.6	0.019	.....	.....	.....
Feb. 27-Mar. 10, 1924..	13	43	1 286.8	0.033	.....	.....	.....
Mar. 11-Mar. 18, 1924..	8	71	671.6	0.106	.....	.....	.....
Mar. 19-Apr. 1, 1924..	14	76	1 338.5	0.057	.....	.....	.....
Apr. 2-Apr. 18, 1924..	17	86.5	1 554.9	0.056	.....	.....	.....
Apr. 19-May 5, 1924..	17	93.0	1 423.2	0.065	.....	.....	.....
May 6-May 19, 1924..	14	84	1 215.2	0.069	.....	.....	.....
May 20-June 9, 1924..	21	111	1 658.6	0.067	.....	.....	.....
June 28-July 14, 1924..	17	102	1 639.9	0.062	.....	.....	.....
July 15-July 23, 1924..	9	206	780.5	0.264	.....	.....	.....
July 24-Aug. 6, 1924..	14	144	1 141.9	0.126	.....	.....	.....
Aug. 7-Aug. 28, 1924..	22	139	1 720.9	0.081	.....	.....	.....
Aug. 29-Sept. 4. 1924..	7	133	575.3	0.231	.....	.....	.....
Sept. 5-Sept. 23, 1924..	19	186	1 680.7	0.110	.....	.....	.....
Sept. 24-Sept. 30, 1924..	8	50	651.8	0.077	September..	4.76	13.00
Oct. 1-Oct. 7, 1924..	7	150	556.0	0.270	Oct. 7.....	8.09	10.39
Oct. 8-Oct. 15, 1924..	8	21	563.0	0.037	Oct. 15.....	9.26	.....
Oct. 16-Nov. 10, 1924..	26	162	1 769.3	0.092	Oct. 30.....	15.31	.....
Nov. 11-Nov. 22, 1924..	12	97	891.8	0.108	Nov. 7.....	10.39	23.30
Nov. 23-Dec. 31, 1924..	39	214	3 164.1	0.067	Nov. 21.....	21.2	54.30
Jan. 1-Jan. 23, 1925..	23	133	1 743.7	0.076	January.....	14.2	45.62
Jan. 24-Feb. 12, 1925..	20	90	1 637.3	0.055	Feb. 12.....	17.2	46.0
Feb. 13-Mar. 9, 1925..	25	78	2 008.2	0.039	March.....	26.8	31.2
Mar. 10-Mar. 18, 1925..	9	130.5	814.3	0.160	.....	14.9	34.1
Mar. 19-Apr. 8, 1925..	21	106	1 568.0	0.068	Apr. 7.....	14.8	27.7
Apr. 9-Apr. 22, 1925..	14	100	1 022.8	0.098	Apr. 22.....	14.5	31.8
Apr. 23-May 8, 1925..	16	102	1 218.4	0.084	May.....	13.9	28.3
May 9-May 29, 1925..	21	111	1 489.0	0.075	.....	21.2	20.3
May 30-June 26, 1925..	28	129	2 196.4	0.059	June.....	14.2	15.4
June 27-July 10, 1925..	14	117	1 233.6	0.095	July.....	13.0	15.0
July 11-July 31, 1925..	21	166	1 858.9	0.089	.....	.....	.....
Aug. 1-Aug. 19, 1925	19	136	1 380.2	0.099	August.....	12.7	24.1
.....	..	.....	.....	.....	.....	10.7	21.6
Totals.....	926	5 957.8	77 417.8	.....	.....	.....	.....
Averages.....	15.7	102.5	1 334.8	0.077	.....	13.69	27.69

of sewage. This figure is comparable to that of 0.077 cu. yd. per 1 000 000 gal. as given in Table 1 after the latter figure is compensated for storm-discharge periods. Suspicion attaches to the experimental quantity because it does not reflect what might be deemed a normal relation to the storm-flow unit of 0.7 cu. yd. per 1 000 000 gal.

The analytic data in Table 1 are not numerous, but define fairly well the character of the grit as regards organic content. The segregation into "Up Stream" and "Down Stream" signifies samples obtained from the pits thus located. From this standpoint there is marked difference between the two deposits, verifying what might readily be anticipated. The grit from the up-stream hopper is rather below the limiting value of 15% fixed by experiment as a rough border line between inoffensive and offensive grit. The material from the down-stream hopper is considerably above the desired 15 per cent. Parenthetically, it may be noted that the moisture content of the two grits indicates that the down-stream material shades off toward sedimentation tank sludge. It is probable that a weighted average of these two samplings would approximate 18% as the volatile and combustible content. In any application or contrast of these figures as a measure of offensive conditions, it is well to recognize that a different sewage might possess characteristics which would materially affect the relation between volatiles and physical nuisance. The grit at the Easterly Plant was used for filling and without offense.

The conclusions derived from a study of the performance record in the light of the anticipations of design are that the type of works herein described functioned in a generally acceptable manner. Future installations of this character would undoubtedly reflect an attempt to reduce the organic content of the down-stream grit. Careful control, however, must be a factor to successful operation. Particularly is this true where, as in this plant, the design contemplates sustained velocities through manipulation of the multiple compartments.

Costs.—In an estimate of worth, the cost enters into the determination. The works herein described cost a total of $1 006 500 subdivided as follows:

Land		$104 500
Plant:		
Bar screens and grit chambers	$278 400	
Conduits, channels, and pipe lines	93 100	
Disinfection	57 000	
Pumping equipment	22 900	
Buildings	93 100	
Parking and miscellaneous	21 700	
		$566 200
Outfall		208 000
Breakwater		127 800
Total		$1 006 500

In terms of maximum daily capacity, the grit chambers and essential appurtenances cost $1 245 per 1 000 000 gal. of sewage. This amount includes screens, grit chambers, buildings, and parking.

Grit Chambers at Westerly Sewage Treatment Works, Cleveland, Ohio

The second of the three major works planned for Cleveland is situated on the lake front at West 58th Street. This plant, built in 1920, includes grit chambers as a preliminary to sedimentation in two-story tanks. Disinfection during the bathing season and dispersion into Lake Erie are essential features of this plant.

The estimated population tributary to this site in 1920 was 311 000, from an area of 9 280 acres.

The grit chambers at the Westerly site are quite similar to the larger installation on the East Side. The grit compartments proper are four in number, in two pairs, divided by a dry-weather channel. The length of each is 50 ft., and the width, 7 ft. The pits are 8 ft. long with bottoms 8.8 ft. below maximum water line for the up-stream pit. The down-stream pit is similar but is 21 in. higher. These chambers are uncovered, a building being placed over the bar screens and the controls. Cleaning equipment includes a monorail loop from which an electrically operated car with clamshell bucket can reach each pit and can dump into industrial cars which run to the dump.

Design.—The design data of the Westerly Works include a capacity of 12 000 000 to 24 000 000 gal. daily for each grit-chamber unit, giving a total maximum capacity of 96 000 000 gal. daily. The average dry-weather flow for this plant is 36 000 000 gal. daily. The planned velocities were from 0.5 ft. to 1 ft. per sec. and the detention period for 50 sec. Pit storage for grit aggregates about 54 cu. yd. Above the pits on the sloping floors is capacity for 48 cu. yd. additional. The total grit storage at this site is, therefore, about 1.06 cu. yd. per 1 000 000 gal. at the maximum or storm rate of flow, and 0.8 cu. yd. if one chamber is out of operation. Also, two-thirds of this capacity is in the up-stream part of the chamber. This contrasts with the provision at the Easterly Works of about 1.38 cu. yd. per 1 000 000 gal., or, roughly, 1 yd. if 3 of the 12 grit chambers were out of operation.

Operation.—Table 2 presents operating records at the Westerly Plant to cover a period of 20 months in 1924 and 1925. These data check closely those given in the record of Easterly operation, Table 1. At the East Side Plant, 926 days' operation yielded 0.077 cu. yd. of grit per 1 000 000 gal. of sewage. At the Westerly Plant, 609 days' operation gave 0.074 cu. yd. per 1 000 000 gal. The maximum deposits are not necessarily susceptible of comparison, but at the larger plant reach a peak of 0.27 cu. yd. per 1 000 000 gal. for a 7-day period, while the Westerly Works showed 0.341 cu. yd. per 1 000 000 gal. during a 6-day period. In passing, it might be noted that, although the greater peaks occur during similar periods at both plants, each plant shows unique features. This is as might be expected from two sites about 9 miles apart.

The few analyses at the Westerly Plant show considerable retention of combustible tar and mash. With these discounted, the organic content in the grit of both pockets is low and unexpectedly similar in both up-stream and down-stream samplings. No ready and certain explanation can be offered of the differences between the analytical results at the two plants.

It is true that the Westerly Plant offers shorter theoretic detention, but, on the other hand, the chambers at the Easterly Plant were operated closer

to their rated capacity than those at the Westerly Plant. It is probable that they provided the greater actual velocity. More probable than any cause appears to be variation in the lengths and arrangements of the sewers tributary to each plant. It is possible that the Easterly Plant, with its larger area and longer sewers, receives its wastes in a condition wherein bottom-dragging grit has entrained more of the results of organic decomposition than appear in the relatively fresher sewage at the Westerly Plant. This is not conclusive but suggests a feature that must be considered in design.

TABLE 2.—Record of Grit Removal, Westerly Plant, Cleveland, Ohio, Sewage Treatment Works.

Period of run.	Days.	Grit removed, in cubic yards.	Sewage flow, in millions of gallons.	Grit, in cubic yards per 1 000 000 gal. of sewage.	Analysis. Date.	Volatile and combustible, percentage. Up stream.	Down stream.
Jan. 1–Mar. 18, 1924 ..	78	28	1 165.1	0.024	.....	.....	.....
Mar. 19–May 29, " ..	72	35	1 056.3	0.033	.....	.....	.....
May 30–June 26, " ..	28	22	326.7	0.068	.....	.....	.....
June 27–Aug. 1, " ..	36	38	406.6	0.093	.....	.....	.....
Aug. 2–Aug. 9, " ..	8	14	85.0	0.155	.....	.....	.....
Aug. 10–Aug. 19, " ..	10	34	99.7	0.341	.....	.....	.....
Aug. 20–Sept. 18, " ..	30	31	347.2	0.090	.....	.....	.....
Sept. 19–Oct. 8, " ..	20	17	258.0	0.066	.....	.....	.....
Oct. 9–Oct. 29, " ..	21	10	231.7	0.043	.....	.....	.....
Oct. 30–Nov. 12, " ..	14	20	192.6	0.104		10.1	*22.0
Nov. 13–Feb. 9, 1925 ..	89	59	1 261.8	0.047	Feb. 2	†36.5	†49.8
Feb. 10–Feb. 20, " ..	11	29	139.3	0.208		*19.3	†38.8
Feb. 21–Feb. 28, " ..	8	5.5	167.1	0.033			
Mar. 1–Mar. 7, " ..	7	20	107.1	0.187		10.1	†51.9
Mar. 8–Mar. 17, " ..	10	6	187.8	0.032	Mar.	†61.7	†71.1
Mar. 18–May 7, " ..	51	26	588.0	0.044		*17.8	*35.4
May 8–May 13, " ..	6	10	68.0	0.147	June	*10.7	*70.5
May 14–June 18, " ..	36	24	396.1	0.061		9.9	6.3
Walworth Run Diversion Sewer Turned in June 22, 1925.							
June 19–July 9, 1925 ..	21	47	632.9	0.074		10.3	12.0
July 10–July 13, " ..	4	17	109.6	0.155	July	*32.1	14.0
July 14–July 23, " ..	10	53	247.1	0.214		10.6	11.4
July 24–July 27, " ..	4	9	98.6	0.092		8.0	10.8
						11.8	8.0
July 28–Aug. 5, " ..	9	29	222.9	0.130		11.0	8.5
Aug. 6–Aug. 20, " ..	15	36	370.8	0.097	Aug.	*22.4	4.8
Aug. 21–Aug. 26, " ..	6	35	146.5	0.239		*17.8	11.2
Aug. 27–Aug. 31, " ..	5	11	112.5	0.098		7.4	6.9
Totals.....	609	665.5	9 025.0	.....	.....	.....	.....
Averages.....	23.4	25.6	347.1	0.074	.....	‡9.9	‡10.4

* Tar present.

† Tar and mash present.

‡ Discarding the presence of tar and mash.

This item of organic content in the grit is one of the essential differences between the operations at the two plants. It is quite clear that the grit chamber at the Westerly Plant rejects fine material which is deposited in

the chamber at the Easterly Plant. This material passes over into the Imhoff tanks and there becomes the lowest stratum of sludge. Thus far, the presence of this silt and fine grit has not affected the tank operation adversely. From the standpoint of fulfilled design, the results at the Westerly Plant are quite satisfactory.

Costs.—Construction costs at the Westerly Plant aggregated $1 047 200, subdivided as follows:

Land		$92 300
Plant:		
Bar gratings and grit chambers	$38 700	
Conduits, channels, and pipe lines	66 700	
Imhoff tanks	600 300	
Disinfection	22 500	
Buildings	78 500	
Parking	16 700	
Sludge disposal	17 500	
Miscellaneous	10 400	
		$858 300
Outfall		$96 600
Total		$1 047 200

Of the total, perhaps $72 900, or 7%, represents the screens and grit chambers and appropriate building and land items. This amount is equivalent to $760 per 1 000 000 gal. maximum daily capacity.

Conclusions

This paper has been prepared to develop a phase of sewage treatment which apparently has received little attention. The writer has tried to establish the possibility of extracting non-offensive solids from the sewage and this to the manifest advantage of subsequent stages in the treatment process. That this can be done at reasonable expense the experiences at Cleveland seem to show. Whether expenditure at the grit chamber or at other point in the plant yields the better results is for local determination. That is a matter of proper balance between cost and results.

It would be idle to expect any standardization from the operation of two plants. Indeed, it should be clear that the interception of grit has perhaps as local a variability as the settlement of organic solids. Earlier ideas in grit-chamber design required the simple enlargement of the main sewer channel, resulting in diminution of velocity and consequent deposit of solids. Duplication of units was included to facilitate cleaning. The Cleveland design provides a considerable number of narrow chambers available for progressive use with increase of flow. By proper design and control the velocities are thus kept closer to those desiderata, and there is possible more exact selectivity of deposited material. The inclusion of pits, in addition to meeting a condition of design, enables economic mechanical cleaning. However, they would be equally convenient for hand-cleaning.

CANADIAN PRACTICE

By F. A. Dallyn,* Esq.

The office with which the writer is associated has had the privilege of reviewing the work of a considerable number of engineering offices and the situations dealt with are almost as diverse as the engineering offices reporting upon them. To the layman a sewerage system represents a concrete idea; to the sanitary engineer responsible for the supervision of municipal works the concrete idea is lost in a multiplicity of contingent circumstances.

The sewage from a municipality which has been sewered many years has a very dissimilar grit content from one in which the sewers have just been completed. The character of the older system also leaves its mark. Some Canadian towns continue old stone drains in the system. There is a further problem in the extent to which storm water enters the system, whether from roof and cellar drains or from the streets. Whether the storm water is brought to the plant or diverted throughout the system by relief sewers also determines the grit content at the disposal works.

Table 3 shows a few figures from several municipalities, giving the quantity of grit per million gallons of sewage.

TABLE 3.

Municipality.	Average flow, in gallons per day.	Cubic feet of grit, per million gallons.	Percentage of organic content.	Remarks.
Toronto, Ont.....	61 800 000 (Max., 142%; min., 68%)	3.20 on average daily flow	20	Storm water by-passed.
Woodstock, Ont....	1 400 000	3.0 in spring	Very offensive	Average velocity, 0.3 ft. per sec.; 24-ft. settling length; baffles introduced to assist in arresting grit.
Guelph, Ont.....	2 000 000 (40% variation)	⅛	Inoffensive, 3 organic	Average velocity, 0.8 ft. per sec.; first section of aeration unit full of sludge from grit chambers; storm flows come into plant.
Kitchener, Ont.....	1 750 000	18 (6 cu. ft. shovelable solid)	Offensive	Average estimated in cross-section area, 0 07 ft. per sec.; sedimentation tank rather than grit chamber;
	750 000	12	Inoffensive	0.8 ft. per sec.; new plant combined system.
Stratford, Ont.....	1 200 000 (25% variation)	4	75 organic	4.15 ft. per sec.; separate sewers only.
St. Thomas, Ont....	1 500 000	3	25 organic	0.6 ft. per sec.; some storm water goes through grit chamber
London, East End, Ont.....	500 000	⅛	.....	0.3 ft. per sec.; domestic separate system.
York Township, Ontario, (Toronto Area).....	4 500 000 (Estimated)	.....	.....	0.4 ft. per sec.; combined system (no data available.)

* Director of San. Eng., Div. Board of Health of Ontario, Toronto, Ont., Canada.

The data in Table 3 do not tell the whole story, because in one of the plants, that at Guelph, periodic storms introduce higher velocities to the grit chambers, which have the effect of flushing the deposits into the forepart of the aeration chambers and also of disturbing the grit that has settled in the outfall sections of the sewerage system itself. The data relative to organic content of the grit given in Table 3 are merely the opinions of the engineers responsible for supervision and design; they are not laboratory determinations. It will also be noted that there is a wide variation due to the effect of the several factors referred to previously.

The problem of designing a grit chamber is not one readily solved by using the weighed average of American, Canadian, or Continental practice. It can only be solved by having due regard to local conditions, taking into consideration the various factors or problems which have intruded into the general plan. The figures in Table 3, showing how grit will deposit at various velocities, are interesting only in regard to those chambers which will operate under well determined conditions of flow, and apply mainly to separate systems in which the character of the sewage is subject only to minor variations. Grit-chamber design for combined systems of small municipalities, without relief sewers, is one of the most intricate problems in connection with sewage disposal, and one which has been almost wholly neglected.

From the writer's observations of combined systems for the smaller municipalities, most of the grit and a large proportion of the organic matter is arrested in the main sewer and outfall sections of the sewer, and the organic matter reaching the plant is very largely a product of the septic action of the material arrested and decomposing in the system. If this observation is correct, the grit chamber as such will function only in times of rainfall. Just what degree of intensity is required to start the grit forward will depend on the system itself. If the grit-chamber velocity is low, a mixture of grit and organic matter will be deposited. It is probable, however, that the grit chambers will be too small and that only the very coarsest grit will be retained, the remainder going into the treatment works proper. This may not constitute a serious problem for some of the older types of treatment, such as plain sedimentation or Imhoff tanks, although it does occasion some difficulty, in that sludge frequently has to be removed by hydraulic jetting or squeegee.

With the new sewage treatment process, that is, activated sludge, the introduction of this organic matter and grit constitutes a serious problem. Its introduction is coincident with greatly increased flows and has a tendency not only to affect the efficiency of the aeration plates by covering them with the gritty portion, but, in addition, tends to increase the quantity of undigested organic matter in the system suddenly, both of which lead to trouble if not corrected by the plant operator.

At one Canadian plant the writer has observed sludge as high as 5 ft. at the inlet end of the activation chamber, due to the effect of storm flows in carrying material forward into the activation tank. Even under these adverse

conditions the tank was operating until arrangements could be made to close down the sections affected, and remove the material by hand, a rather expensive operation for the particular works.

Grit-chamber design for combined systems, therefore, is a problem that cannot be solved in accordance with what is now recognized as accepted practice. The writer believes that in the future some type of storm tank will be used into which these excess flows with a grit content can be diverted, and that such grit chambers as ordinarily function in the system will be designed to meet the specific operating problem for the regulated or determined flows of the particular system under review.

Just how large these storm tanks should be the writer is not prepared to state. Experiments are being made at Kitchener and at York Township. The sewers in both these localities discharge into watercourses which must be kept free of mounds of sludge, during the summer season. In these plants the storm-water tanks will have a capacity somewhat less than the ordinary grit-chamber design for the maximum flows. For the ordinary storm, the retention will be 20 to 30 min. The tanks also are arranged to receive the accumulated grit and organic matter from the smaller service grit chambers, and, in addition, will receive all the surplus sludge during the winter months and a lesser quantity during the summer season. Dorr equipment has been introduced in the storm tanks for facilitating the removal of the arrested material. In one instance a Dorco pump will be used for removing the sludge, and in the other an ordinary sludge pump which has been in use on the system will be continued for this special purpose.

BRITISH PRACTICE IN THE DESIGN OF GRIT CHAMBERS OR DETRITUS TANKS

ARTHUR J. MARTIN,* ESQ.

This subject at first sight might appear to afford little material for a paper, for more than one author with a wide experience of British sewage works has stated that there is little evidence of design in detritus tanks. This statement is hardly fair to British engineers, most of whom doubtless have evolved in their own practice types of tank which they find to meet their requirements. However, little information has been published in England on the subject of grit chambers, and English textbooks on the disposal of sewage are all but silent with regard to them. This is the more surprising considering that the large towns, with hardly an exception, are sewered on the principle of the combined system, and that the quantity of road detritus which finds its way into the sewers is often considerable. At Manchester, England, it is reported, as much as 300 tons of detritus sometimes reaches the outfall in 24 hours.

Separate provision for the interception of grit is by no means general. Where the sludge is pressed, the presence of a high proportion of mineral matter facilitates the separation of the liquid; and in such cases grit chambers are often omitted. The latest Royal Commission on Sewage Disposal (1898-1915) thought it best in the case of precipitation or sedimentation tanks, to settle out "a considerable proportion" of the mineral detritus before allowing the sewage to enter the tanks. For septic tanks it was of the opinion that a "fairly thorough preliminary settlement of grit" was generally desirable. For very small works, however, it considered that "the provision of grit or detritus tanks which require constant sludging may necessitate more labor than would otherwise be the case, and in such circumstances it may be more economical to allow everything to go forward into the septic tank."†

In a discussion before the Institution of Civil Engineers, Mr. Midgley Taylor‡ expressed the opinion that:

"The removal of the dry-weather grit content was unnecessary. He [Mr. Taylor] had yet to learn that ordinary domestic sewage had any appreciable quantity of grit in it, and he suggested that the whole of the contents of the sewers, after passing through the screen, should be led directly into the Dortmund tank."

The witnesses who are cited by the Royal Commission in its "Summary of Evidence on Certain Questions" are almost unanimously of the opinion that heavy detritus should first be removed.§

* Chartered Civ. Engr. (Martin & Roberts), London, England.

† Fifth Rept., Royal Comm. on Sewage Disposal, 1908, p. 19.

‡ *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, pp. 121–122.

§ Fifth Rept., Royal Comm. on Sewage Disposal, 1908 Appendix II, pp. 32–33.

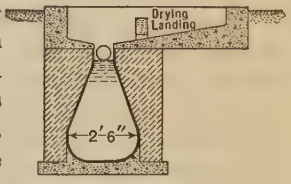
Where special grit chambers or detritus tanks are not provided, the inlet end of a septic or sedimentation tank is often weired off from the remainder to serve the purpose.

The American practice of designing grit chambers with a view to preventing the deposition of organic solids is practically unknown in England, capacity, rather than velocity, being the usual basis of design. Recent British practice in this respect is governed by the requirements of the Ministry of Health, based on the findings* of the Royal Commission on Sewage Disposal, that "there should be two or more detritus tanks below the screening chamber. The capacity of each tank should be about $\frac{1}{100}$ of the '(daily)' dry-weather flow."

The capacities of the detritus tanks in use in different towns vary widely, ranging from less than 0.5% to 20% or more of the daily flow. In some cases capacities up to 30% are reported; but the nomenclature of sewage works is not standardized, and some of the tanks in question would be more properly described as sedimentation tanks.

Detritus tanks are usually rectangular in plan, often with comparatively little difference between their length, breadth, and depth; but deep circular tanks are sometimes used. Most rectangular tanks have sloping floors, the depth being as a rule greatest at the inlet end.

At Kingston-on-Thames a small rounded chamber is formed by the side of the channel along which the sewage passes on its way to the precipitation tanks, and the grit in process of settlement in this chamber is washed by the whirling motion set up. Tyndale, for many years Sanitary Adviser to the War Department, designed a grit chamber having a cross-section like that of a conical flask, through the narrow neck of which the floating solids pass on unchecked to the septic or sedimentation tank (Fig. 5).†



CROSS SECTION OF GRIT CHAMBER
FIG. 5.

In small works, the writer has used a detritus chamber in the shape of an elongated pyramid, one of the sides of which is formed by the surface of the sewage. The latter enters at the apex of the pyramid and flows on to the base in a stream which widens and deepens as it goes. Along the lower side of the base there is a slot through which the deposited solids drop into a chamber beneath. The passage of the solids through the slot is assisted by an induced flow into and through the lower compartment set up by the sewage as it enters the upper one. The chamber in question was expressly designed to intercept the heavier organic solids, the grit being previously caught in a small pit.

A grit chamber more in accordance with American practice was recently built by Jameson at Pretoria, South Africa, and was described by him in a paper before the Institution of Civil Engineers in 1923. This chamber, which is 27 ft. long by 6 ft. wide, was originally formed with a dished bottom. It was found that sludge accumulated in the depression and that septic

* Fifth Rept., Royal Comm. on Sewage Disposal, 1908.
† The War Department Drainage Manual, 1921, p. 52.

action took place. To prevent this the floor of the chamber was raised by successive installments to a level flush with the outlet weir, reducing its holding capacity to zero. The normal depth of the sewage passing through the chamber is about 4 in. and the grit is deposited along the center line of the floor.*

Grit chambers are commonly formed in conjunction with screening chambers, the screens being built of parallel bars, and kept clear by mechanically driven rakes. Fine screens are practically unknown in England.

In large works, in which the grit chambers are often sunk deep in the ground, the detritus is usually removed by dredges or grabs. In smaller works it is scooped out by hand or lifted by means of a chain pump. Where fall permits, a cleansing valve is sometimes provided. For the purpose of running off the liquid contents of the chamber without interference by grit, the writer frequently uses a vertical stoneware pipe built into the wall, having a narrow slot on the side next the chamber and a lift-up valve in the bottom.

At the Saltley works of the Birmingham, Tame, and Rea Drainage Board, the sewage, before entering the detritus pit, passes through a grid formed of bulb bars, 7 in. apart, sloping upward in the direction of the flow at an angle of about 30° from the invert of the sewer.

The detritus pit is V-shaped in section, 240 ft. long, 14 ft. wide at the surface, and 19 ft. deep. At the bottom it is just wide enough to allow the dredger buckets to pass between the walls. The dredger (Fig. 6) is similar to that at Shieldhall, Glasgow, and consists of a traveling bucket elevator supported on a four-wheeled carriage, running on a standard gauge railway supported by cast-iron pillars, thus leaving an open space from end to end of the pit into which the liquid from the sludge trucks may drip without causing a nuisance. The arm carrying the dredger buckets is 40 ft. long, supported on trunnions about 13 ft. from the top, and the buckets, which are perforated to allow the water to escape, are carried on an endless chain which passes around drums placed top and bottom. To the bucket arm is connected a hopper and chute to convey the detritus into the sludge truck, which travels behind or in front of the dredger as it moves backward and forward. The dredger is worked by a single motor of about 15 h.p., which collects current from wires alongside the pit.

The pit has a dry-weather capacity of about 1 500 cu. ft. per 1 000 000 gal. treated, and the sewage flows through it at the rate of about 3 in. per sec., depositing sand, slack, rags, paper, fæces, etc., in its course. When the sludge truck is filled it is hauled to the tip by a steam locomotive.

Prior to 1902 the detritus pits were limited to portions of the sedimentation tanks, and there was a stationary dredger to lift the sludge into wooden troughs, which were more or less permanent near the dredger, but as the distance from the dredger increased they were constantly lifted and relaid in various directions toward land capable of receiving the dredgings.

Subsequently, and until the new dredger was installed, the first bay of the roughing tank was used to arrest detritus, and a Priestman grab was

* *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, pp. 90–91.

FIG. 6.—VIEW OF DETRITUS SETTLEMENT CHAMBERS, KNOSTROP, LEEDS, ENGLAND.

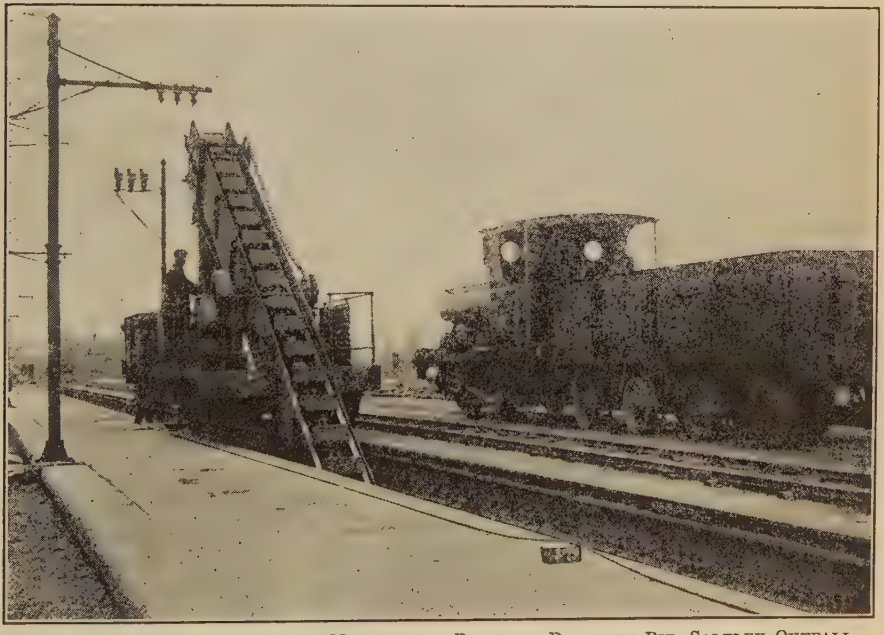


FIG. 7.—ELECTRICALLY DRIVEN MECHANICAL DREDGER, DETRITUS PIT, SALTLEY OUTFALL WORKS, BIRMINGHAM, ENGLAND.

TABLE 4.—Removal of Detritus at Cole Valley (Ashold) Works.

Year.	Annual volume of sewage, in gallons.	Quantity of Detritus, in Cubic Yards.			Total annual charge, for attendance and maintenance, including interest on capital outlay.	Cost of Removal per Cubic Yard of Detritus.*					Total per million gallons.
		By screens.	By dredger.	Total.		Electric power at 0.6d. per unit.	Attendance and removal to tip.	Repairs.	Interest on capital outlay.	Total per cubic yard.	
1906	818 000 000	276	1 616	1 892	£287 16s. 11d.	3.06d.	5.77d.	1s. 5.82d.	9.80d.	3s. 0.45d.	7s. 0.3d.
1907	857 000 000	253	1 501	1 754	204 9 9	3.06	6.22	0 8.13	10.57	2 3.98	4 9.3
1908	900 000 000	273	1 379	1 652	206 11 4	3.06	6.61	0 9.11	11.23	2 6.01	4 7.1
1909	941 000 000	262	1 501	1 763	289 19 3	3.06	6.19	1 7.70	10.52	3 3.47	6 1.9
1910	980 000 000	308	1 499	1 807	188 19 3½	3.06	10.92	0 0.85	10.26	2 1.09	3 10.3
1911	1 018 000 000	323	1 020	1 343	257 6 5	3.06	14.38	1 2.74	13.80	3 9.98	5 0.7

* Measured after water is separated from it without pressure.

utilized to lift the detritus from the tank into dump wagons, drawn by horses to the dump.

At the two smaller works operated by the Board—Sutton Coldfield (Tyburn) Works and Cole Valley (Ashold) Works—the mode of arresting the grosser solids is slightly different from that used at Saltley. The detritus pit at Ashold has a capacity of 780 cu. ft., and is controlled by a stationary dredging appliance so arranged that the buckets drop the detritus into a hopper, then on to a horizontal trough fitted with a worm, which conveys the material to a dump wagon stationed outside the building.

Mr. J. D. Watson, to whom the writer is indebted for these particulars, states that the stationary form of dredger is not so satisfactory as the other, because the breaking of a chain or the renewal of a sprocket wheel necessitates turning the sewage into the by-pass and emptying the pit before repairs can be effected, whereas the Saltley dredger may be lifted entirely out of the sewage at a moment's notice.

Table 4 gives the quantity of detritus removed from the Ashold Works and the cost of its removal over a period of six years.

Fig. 7 shows the detritus tanks, electric traveling crane, gantry, and Priestman grab at the Leeds Corporation Low-Level Works, at Knostrop.

The design of grit catchers, screens and storm-water tanks formed the subject of a paper read by Mr. F. C. Temple, of Jamshedpur, India, at the International Conference on Sanitary Engineering held in London, in 1924.*

* *Transactions*, International Conference on San. Eng., 1924, pp. 190–203.

GRIT CHAMBERS IN GERMAN SEWAGE PURIFICATION PLANTS

By Karl Imhoff,* M. Am. Soc. C. E.

In large German plants, deep hopper-bottomed grit chambers predominate. This type of construction originated in England and is very practical for the mechanical removal of the grit, for instance, by bucket elevator. Its disadvantage is the poor quality of grit produced, which is mixed with offensive organic sludge.

Since 1906, long narrow flat-bottomed grit chambers have been used in combination with two-story tanks. The purpose is to keep the offensive organic matter in motion by an established velocity (about 1 ft. per sec.) and to flush it into the digestion chambers of the two-story tanks. This procedure ordinarily permitted the settlement of clean and odorless grit which could be used as filter sand on the sludge drying beds. If the grit was occasionally unsatifactory because of a temporary low velocity, experience showed that the quality could be improved by stirring under the flowing sewage before removal. These long narrow chambers have drain pipes for drying the sand, and permanent tracks to accommodate small dump cars for the removal of the dry grit by hand. Moveable clamshell buckets are used in large plants.

In some plants two-story grit chambers, with the lower compartment for the collection of the grit, have been tried. Unfortunately, the grit collected is not much cleaner than that in any other deep grit chamber. Evidently, organic sludge is deposited from the part of the sewage that passes through the lower chamber.

* Chf. Engr., "Ruhr-Verband," Essen, Germany.

DISCUSSION

JOHN F. SKINNER,* M. AM. SOC. C. E.—In discussing this subject, the speaker will present some facts and observations concerning the design, construction, and operation of the grit chambers at the Rochester, N. Y., plants. These plants will be first briefly described, after which the general subject will be discussed.

Data are available (1925) from three disposal plants at Rochester, N. Y., as follows: The Irondequoit Plant, designed by the late Emil Kuichling, M. Am. Soc. C. E., consists of coarse racks with 3-in. openings; six parallel grit chambers, 10 ft. wide, 9 ft. deep, and 90 ft. long, each followed by a 12-ft. Reinsch-Wurl screen (four now installed); influent and effluent channels, and float-regulated gates which admit the screened sewage to twenty Imhoff tanks (ten now installed); eighty sludge beds (forty now installed); 66-in. lock-bar steel outlet pipe, 9 000 ft. long, extending 7 000 ft. into Lake Ontario in 50 ft. of water; at present receiving about 36 500 000 gal. per day of combined sewage from 275 000 persons.

The Brighton Plant, designed by the speaker, consists of three parallel grit chambers, each 4 ft. wide, with a deep pocket; fine racks with $\frac{1}{8}$-in. openings; penstock; two Imhoff tanks (one now installed); 2 acres of sprinkling filters (1 acre now installed); four sludge beds (two now installed); four final sedimentation basins (two now installed); and two 12-in. outlet pipes, 1 100 ft. long, into Irondequoit Creek; at present receiving about 1 500 000 gal. per day of sanitary sewage from 10 000 persons.

The Charlotte Plant, designed by the speaker, consists of two parallel grit chambers, each 4 ft. wide and 40 ft. long, with an end pocket; fine racks, with $\frac{1}{8}$-in. openings; 6 and 7-in. air-lifts; circular Imhoff tank; sludge beds; and $\frac{12}{17}$-in. outlet pipe, 300 ft. long, to the Genesee River; operating house with electrically operated air compressors in triplicate; at present receiving from 300 000 to 1 000 000 gal. per day of combined sewage from a population of from 3 000 to 20 000, as the plant is in a suburb which includes the city's principal summer resort. Fig. 8 (*a*) to (*d*) shows sections of the grit chambers of the plants mentioned, together with a cross-section of the grit chamber of the proposed Maplewood Plant. Available records from these plants will be used for illustration.

I.—Method of Approach.—At the outset the following should be considered:

(*A*) Sources of grit in sewage: (1) In sanitary sewage, grit may come from the bathing of people and washing of animals; from erosives and erosive cleansers; from the washing of cellars and floors which contain dust and earth carried in by the feet and clinging to commodities; from the washing of vegetables and foodstuffs and the preparation of food; from silt which enters the sewers through openings and poor joints and with ground-water from the surreptitious drainage of trenches and excavations into the sewers; (2) in storm water, grit may come from street wash; from dust blown on roofs; from erosives; and from the draining of excavations; and (3) in com-

* Cons. Engr.; Deputy City Engr., Rochester, N. Y.

bined sewers and storm overflows, all the sources just mentioned may contribute grit.

(*B*) Character of grit: Physical and chemical properties. Properties such as organic content, fineness, and specific gravity are exhibited in Table 5.

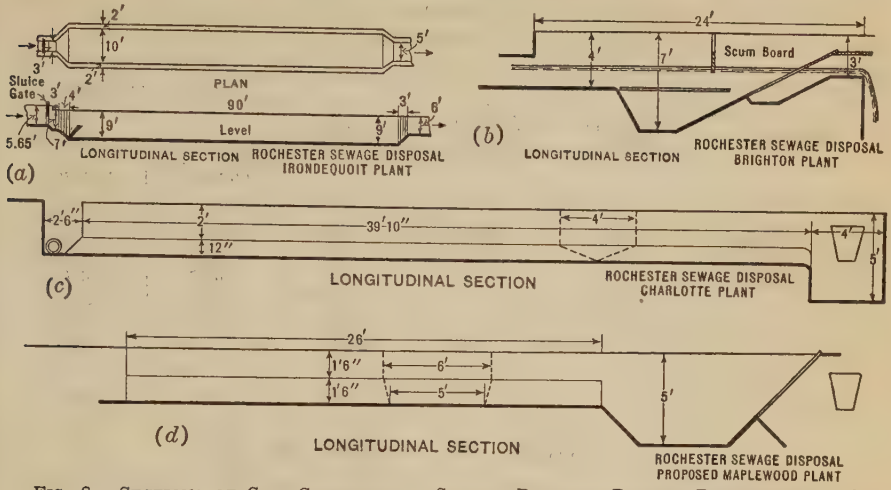


FIG. 8.—SECTIONS OF GRIT CHAMBERS OF SEWAGE DISPOSAL PLANTS, ROCHESTER, N. Y.

Of the clean grit from the Irondequoit grit chamber, 1 cu. ft. drained and packed weighs 85 lb. The specific gravity of the mass is 1.36. With voids assumed at 35%, the average density of the material will be 2.08, as shown in Table 5 (*A*) for mixed samples. This grit, although from a combined sewer system, shows the same berry and grape (raisin) seeds that are found in large quantities in the Imhoff sludge at the Brighton Plant which treats the sewage from a separate system. These seeds, together with particles of coal, are the principal objects which give "loss on ignition". Neither the seeds nor the larger particles of coal pass the 20-mesh sieve, which fact readily accounts for the 37% loss on ignition above this size and only 15% below it. It will be further noted that the specific gravity of the finer material is 2.23, whereas that held on the 20-mesh sieve is 1.45.

Only a little of this material is putrescible, but at present considerable quantities of grain are found in the grit, the probable source being the wastes from home-brewing processes. This material is found in layers near the downstream end of the deposit in the Irondequoit grit chambers and at about one-half the depth of the deposit. A few days after de-watering the chamber, in warm weather, this material is foul smelling when uncovered.

(*C*) Quantity of grit: Certain substances and objects will be deposited in grit chambers at low velocity, and at slightly higher velocity will be carried on and caught on the screens. The total of such material at the three plants is shown in Table 6.

It will be noted that wide variations in the quantity of grit collected occur from year to year, due to the fact that most of this material is brought down when the velocity in the sewers is increased by rain and melting snow. It,

therefore, varies with the meteorological conditions, as they produce greater or less scouring velocities.

TABLE 5.—MECHANICAL ANALYSIS OF GRIT FROM DETRITUS CHAMBERS, IRONDEQUOIT PLANT, ROCHESTER, NEW YORK.

(*A*) JUNE 3, 1919.

Material.	Specific gravity.	Loss on ignition, percentage.	Mesh per linear inch.	Weight, in Grammes.	
				Passing each sieve, retained on next.	Total amount passing.
Mixed samples.....	2.08	23.3	4	33.20	190.18
Held on 20-mesh sieve..	1.45	37.0	10	38.95	156.98
Passing 20-mesh sieve..	2.23	15.0	20	30.36	118.03
.....	....	....	30	30.42	87.67
.....	....	....	40	30.20	57.25
.....	....	....	50	9.63	27.05
.....	....	....	80	6.65	17.42
.....	....	....	100	10.77	10.77
Total.....	....	.....	...	190.18	190.18

(*B*) NOVEMBER, 1921, AND JANUARY, 1922.

Mixed samples, Nov. 28, 1921, and Jan. 30, 1922.....	1.77	....	4	32.2	160.3
.....	....	....	10	40.5	128.1
.....	....	....	20	27.5	87.6
.....	....	....	30	16.5	60.1
.....	....	....	40	14.5	43.6
.....	....	....	50	27.5	29.1
.....	....	....	80	0.8	1.6
.....	....	....	100	0.8	0.8
Total.....	....	.....	...	160.3	160.3

The quantity of grit deposited is commonly given as so many cubic feet, cubic yards, or pounds per million gallons of sewage. This may be a convenient statement, but in determining the capacity for grit in a proposed design it must be remembered that this material does not come to the plant with each 1 000 000 gal. of sewage, but that it settles and is stored in the sewers (if their dry-weather velocity is small) until a storm flow occurs to wash it to the treatment plant. Even if the sewers have self-cleansing velocities at all times, the great mass of grit is washed in from the streets during storms and reaches the plant with the storm flow. The capacity for storage of grit, therefore, must be based on the quantity that may be brought down by the maximum of storms which may occur in the interval between assumed reasonable periods of cleaning.

(*D*) Deposition of grit: sedimentation, hydraulic value, and critical velocities.

Data collected by Allen Hazen, M. Am. Soc. C. E.,* have been used by the speaker in studying several proposed designs for grit chambers. These data,

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 63.

somewhat amplified, together with certain derived information, are given in Table 7, exhibiting the sedimenting characteristics of grit.

Table 7 may be used to determine a tentative length of grit chamber which will allow settlement of the various sizes by multiplying the corresponding periods in Column (6) by the proposed depth of the chamber. This is based on a velocity of 1 ft. per sec. As the experiments on which Table 7 was based were made in still water it is apparent that, in a flowing stream, eddies will occur which will lengthen the path of a particle of grit and thus increase its velocity.

TABLE 6.—Quantities of Grit and Screenings Removed from the Rochester, N. Y., Disposal Plants, in Cubic Feet per Million Gallons.*

Year.	Screenings:			Grit:		
	Irondequoit.	Brighton.	Charlotte.	Irondequoit.	Brighton.	Charlotte.
1916	....	0.86†	....	....	4.29†	....
1917	4.26†	0.66	....	1.99†	0.73	....
1918	6.11	0.42	....	1.93	....	....
1919	6.55	0.36	....	2.67	....	....
1920	4.58	....	....	2.58	....	....
1921	3.60	2.77	....	2.40	....	....
1922	3.94	3.88	....	3.22	....	....
1923	5.13	5.93	3.12	3.07	....	....
1924	5.15	4.98	3.13	2.56	....	....
1925	5.35‡	5.36‡	2.27‡	3.02‡	....	....
Weighted mean...	4.97	2.81	2.89	2.60	....	....

* The Irondequoit Plant began operation in March, 1917, the Brighton Plant on March 1, 1916, and the Charlotte Plant in November, 1921.

† 10 months.

‡ 9 months.

Another disturbing element is the commotion incident upon entry to a chamber through a gate or along a channel which approaches at an angle or changes in direction. In the large grit chambers at the Irondequoit Plant the deposit commences 15 or 20 ft. along the 90-ft. chamber from the entry end. A velocity of 1 ft. per sec. is satisfactory in classifying material and in causing grit to settle and allowing most of the organic solids to pass on. Particles of relatively large size will also settle in the chamber even if their specific gravity is considerably less than the 2.65 mentioned in Table 7.

For large plants where, say, 20 000 000 gal. daily may pass through a single chamber, a width of 10 ft. and a depth of 4 ft. will allow 1 ft. of deposit and still maintain a velocity of 1 ft. per sec. Broad tanks require less depth to maintain a given velocity and the length of the tank is proportional to the depth for a given cross-section. The tank must not be too broad for convenient cleaning, nor for the maintenance of uniform velocity in the cross-section.

For small plants where 2 000 000 to 3 000 000 gal. daily will pass through the chamber, a channel 4 to 6 ft. wide, and with the depth varied by a suitably placed weir, either fixed or adjustable, will readily maintain a desirable velocity. In yet smaller plants, more uniform velocity can be maintained in a chamber

4 ft. wide, with a V-shaped bottom formed with slopes of 1 on 2. In such plants a deeper pocket at the end of a uniform channel is a valuable feature, for deposits of coarser grit in the up-stream sections of the channel may be washed down by a hose stream into the pocket, thus maintaining a relatively clean channel of constant cross-section and, at the same time, collecting the grit at one point for removal.

TABLE 7.—Sedimenting Characteristics of Grit.

Diameter of particle, in millimeters.	Hydraulic Value of Particle at 50° Fahr.		Weight of sand particles of specific gravity 2.65, in grammes.	Number of particles of sand per gramme.	Time required to settle 1 ft., in seconds.
	Millimeters per second.	Feet per second.			
(1)	(2)	(3)	(4)	(5)	(6)
1.00	100	0.328	0.001387	720	3.0
0.80	83	0.272	0.000710	1 408	3.7
0.60	63	0.207	0.000299	3 742	4.8
0.50	53	0.174	0.000173	5 882	5.7
0.40	42	0.138	0.000089	11 262	7.2
0.30	31	0.105	0.000037	26 684	9.5
0.20	21	0.069	0.000011	90 090	14.5
0.15	15	0.049	0.0000047	213 675	20.4
0.10	8	0.026	0.0000014	724 652	38.4
+0.095	7.6	0.025	0.0000012	.....	40.0

In all cases, dual or multiple chambers are desirable, both for convenience in cleaning, efficiency, and economy, because in storm flows two or more may be put into service thus maintaining a proper velocity.

For a large chamber like those at the Irondequoit Plant, the grit begins to build up 15 or 20 ft. from the up-stream end, and as this bar of grit increases in height and reduces the cross-section of the stream, the finer grit is carried along at the higher velocity and deposited farther down stream after it has had time to reach the deeper bottom.

In general, the coarser material is deposited up stream, but fine material is always found along the bottom beneath the coarse material in the down-stream half of the chambers. Probably as the detritus collects in the channel more and more of the finer material is carried along into the Imhoff tanks by the increased velocity. Indeed, at the Irondequoit Plant, the noticeable presence of grit on the brushes of the Reinsch-Wurl screens is one of the symptoms which indicate when the grit chambers require cleaning.

A study of critical velocities at which various sized particles will move or be deposited would be interesting but, in general, the grit to be removed is so much coarser than that which is entangled in organic solids and so much heavier than the organic solids that essential classification may be effected if a velocity of about 1 ft. per sec. is maintained for a sufficient period.

(*E*) Grit chambers are desirable:

1.—Under various conditions, such as: (*a*) At all times in treatment plants receiving sewage from a combined system. If, by means of grit chambers,

4 to 6 cu. ft. of grit per 1 000 000 gal. can be removed from combined sewage, the inclusion of grit chambers is justified in the design of a plant; (*b*) in plants treating sanitary sewage where there is danger that the heavy solids may clog the sludge pipes of deep tanks. Some grit will of necessity go on into the Imhoff tanks, but any large quantity of heavy sand will render the drawing of sludge difficult; (*c*) in plants where mechanical devices, such as pumps, water wheels, and fine screens cleaned by brushes, are liable to excessive wear if the grit is not removed; (*d*) where a long outlet pipe discharges the sewage or effluent at a low velocity and there is consequently a liability of sedimentation in the pipe and reduction of its capacity; (*e*) at storm-water overflows,* where it becomes necessary to reduce the silting up and pollution of streams from this source.

2.—To remove coarse and heavy material as fine as economically advisable with the minimum of putrescible organic matter. A small quantity of grit necessarily continues on through the preliminary processes, entangled or enveloped in organic solids from which it becomes disengaged in the process of digestion in the Imhoff tanks. The quantity and size of this material have been determined in three instances as shown in Table 8.

The quantity of so-called grit in the samples of Imhoff sludge from the Irondequoit Plant varies, as exhibited in Table 8, from 13.8% of the solids (dry basis) at the first drawing in the spring to 3.8% for the drawing in midsummer. This "grit" is to some extent made up of seeds and coal, as is indicated by the "Loss on ignition". This loss is greater in the larger sizes, the same as in grit-chamber material (Table 5).

The Irondequoit samples indicate that after ignition only 10% of the grit from the grit chambers is below the "passing 50, retained on 80-mesh" size, whereas in the "grit" from the Imhoff sludge the same size amounts to 65% of the residue after ignition. This gives a practical limiting size of 0.20 mm. which, if it has a specific gravity of 2.65, may be expected to settle 1 ft. in about 14.5 sec. in still water. The grit that passes after such removal has been found to cause no difficulty in an Imhoff tank and can be discharged readily with the sludge.

II.—Design of Grit Chambers.—

(*A*) Conditions of service:

1.—If a territory is thoroughly built up and the population fairly constant the conditions will be relatively uniform at the plant and the design may be definitely fixed.

2.—If the territory is new and increasing in population the engineer must discount the future in his design, building a number of units or providing for their construction later.

3.—If the territory presents a widely varying population, such as a summer resort, with a large number of cottagers and crowds of transients on pleasant days, these wide variations must be considered in the design for it will be desirable to maintain an approximately uniform velocity through the grit chambers at all times.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 438.

(*B*) Type of chamber:

1.—Long chambers of uniform section are economical to construct and may be laid out in parallel, with common partition walls. They are convenient to clean and may be spanned by travelers running on rails on or above the longitudinal walls.

TABLE 8.—Mechanical Analysis of Grit from Imhoff Tank Sludge.

(*A*).—Brighton Plant. 1917 (300 Grammes of Wet Sludge).

Characteristics of Wire Mesh Sieves.					Weight, in Grammes.	
Mesh per linear inch.	Diameter of wire, in millimeters.	Size of Square Opening.			Passing each sieve, retained on next.	Total quantity passing.
		Millimeters.	Inches.	Fraction.		
4	.....	.....	.....	....	1.06	2.46
10	.....	.....	.....	....	0.31	1.40
20	.....	.....	.....	....	0.10	1.09
30	.....	.....	.....	....	0.07	0.99
40	.....	.....	.....	....	0.19	0.92
50	0.212	0.2960	0.01165	1/86	0.28	0.74
80	0.140	0.1775	0.00699	1/143	0.24	0.45
100	0.099	0.1550	0.00610	1/164	0.19	0.22
200	0.051	0.0760	0.00299	1/334	0.03	0.03
Total.....	.....	.....	.....	....	2.47	....

(*B*).—Irondequoit Plant, 1922.

First Drawing of Sludge in 1922. 60 Grammes of Dried Sludge Yielded 8.3 Grammes of Heavy Solids, "Grit."				Drawing of Sludge, July 7, 1922. 138.60 Grammes of Dried Sludge Yielded 5.2 Grammes of "Grit."	
Mesh per linear inch.	Weight, in Grammes.			Weight, in Grammes.	
	Passing each, retained on next.	Total passing.	Loss on ignition.	Passing each, retained on next.	Total passing.
4	0.0	8.3	...	0.6	5.2
10	0.4	8.3	0.4	1.0	4.6
20	0.4	7.9	0.3	1.0	3.6
30	0.5	7.5	0.5	0.4	2.6
40	0.6	7.0	0.4	0.4	2.2
50	4.4	6.4	2.9	1.1	1.8
80	0.3	2.0	—0.1*	0.2	0.7
100	1.7	1.7	0.1	0.4	0.5
200	0.0	0.0	—1.4*	0.1	0.1
Total.....	8.3	8.3	3.1	5.2	5.2

* Larger sizes broken down by burning caused an increase in weight of these sizes.

2.—Short chambers with deep pockets have the advantage of collecting grit at one point for removal, the mean velocity in the pocket and in the channel of approach being inversely as the depth of water. Such pockets call for careful design to avoid, on the one hand, eddying and scouring and, on

the other, a stratum of dead water in the bottom of the pocket over which the sewage may flow from the channel of approach without appreciable change of velocity. In the latter case the channel would have to be designed as a long one, using the pocket merely for storage of the deposited grit. This uniform velocity may be maintained over the pocket by a false bottom of slats through which the deposited grit may fall.

3.—(*a*) Long chambers of increasing cross-section are suggested as a type which may be useful where it is desired to collect fine grit. This form will assist in classifying the material for larger heavy particles will settle near the entry end and the finer, progressively along the channel as the velocity decreases. The depth should be constant and the channel become wider down stream. (*b*) Tanks of uniform width, but with greatest depth near the entry end, growing shallower down stream, usually have a reverse current along the bottom for some distance from the deep end. This serves to move the fine grit which has settled in the shallow end back to the deep section.

4.—The position of entry gates and coarse racks and the means of baffling at entry so that the stream will smooth out as soon as possible, are important, as well as convenient in making repairs or replacements. A type of gate which will admit water over the full width of the chamber is preferable to a small gate. Coarse racks placed in each chamber a short distance up or down stream from the gate may be inclined for convenience in raking and if up stream from the gates, the entire rack may be slid down grooves in the side walls so as to be readily removed for repairs. If down stream from the entry gate it will be exposed when the chamber is dewatered, and may act as a baffle. Baffling to break up a rapid thread in the stream can best be accomplished by trial.

The deposit in the Irondequoit grit chamber commences 15 or 20 ft. down stream from the inlet. It builds up to about 3 ft. depth, decreasing slightly down stream. In September, 1925, the speaker suggested that a 12-in. plank baffle with its top inclined down stream be set across the bottom of one of the chambers at its up-stream end, the purpose being to deflect the flow upward, obviating the scour on the bottom, and causing an eddy in which deposit might occur. The baffle was placed in No. 2 Chamber, which was operated in parallel with No. 3 Chamber and under identical conditions during a period including a week of heavy rains. When the chambers were dewatered the utility of the baffle was at once apparent. The deposited grit started at the baffle and even gathered in the corners up stream from it where formerly the bottom had been scoured clean. The operator at the plant further reports that 116 cu. yd. of grit were removed from this chamber, whereas the usual removal is from 66 to 70 cu. yd. One previous instance only gave a removal of 96 cu. yd. This simple baffle, therefore, has added about 50% to the efficiency of the grit chamber.

III.—Location of Grit Chamber in Flow Line of Treatment Plant.—The grit chamber should be up stream from fine racks or screens, for the reason that the heavy grit will settle out at a relatively high velocity and thereby relieve the sewage before subsequent treatment wherein the grit might cause damage or difficulties.

IV.—Methods of Removal of Grit.—

(*A*) Small plants handling sanitary sewage only will produce but a small quantity of grit, which can be shoveled into pails. When this quantity has become somewhat greater, steel cans 17 in. in diameter and 24 in. deep, with bales, can be efficiently handled with a chain tackle and sidewalk ash hoist. This hand work requires de-watering of the chamber.

(*B*) In large plants, power may be used with dipper or clamshell buckets, or bucket and belt, or screw conveyors. These may be used after stopping the flow through the chamber, but do not necessarily require de-watering. Clamshell and dipper buckets are likely to injure the bottom masonry. For this reason the clamshell formerly used at the Irondequoit Plant has been discarded and the body of a Koppel Industrial Railway car is lifted from its truck and lowered into the chamber by the shovel. After filling by hand the car body is replaced on its truck and when a train is made up, a storage battery locomotive hauls it away. This requires de-watering of the chamber, which in a few days drains satisfactorily.

At Rochester the electric power is generated hydraulically from the fall of the sewage to the level of Lake Ontario. About 150 h.p. is available at all times. In Syracuse, N. Y., clamshell buckets the width of the chamber ride on rails flush with the bottom of the chamber and scrape the bottom clean. In small municipal plants where there is usually considerable hand labor available, an expensive mechanical equipment requiring a skilled operator, as well as repairs and maintenance, will often not justify its cost.

In a large plant, a bucket and belt conveyor running on tracks on the longitudinal walls of the chambers and provided with a transfer table offers an attractive solution. A sand sucker and a vacuum ejector offer interesting methods for investigation.

(*C*) Continuous removal of grit has not been generally practical as the grit is not deposited uniformly or continuously. Continuously moving scrapers connected by linked chains running longitudinally and rising on a ramp at one end of the chamber have been used, and a curved or semi-circular cross-section for the bottom of the chamber with scrapers operating transversely has been suggested.

V.—Final Disposition of Grit.—Grit is generally used for filling and is often in demand. Due to its porosity it is valuable for mixing with soil growing certain garden crops, notably, cucumbers, squash, melons, and tomatoes. It is also useful as a top dressing for dirt or gravel paths and drives for light or occasional traffic. In this respect, it is similar to coal cinders.

A. F. Macallum,* M. Am. Soc. C. E.—Apparently, little attention has been given to the design of the concrete grit chamber. No doubt the reason is that grit chambers have not always been considered necessary with different types of sewage disposal plants except with the activated sludge treatment.

Another point that has not been discussed is the character of road surface from which the sewers bring the surface drainage to the disposal works,

* Commr. of Works, Ottawa, Ont., Canada.

which will affect the size of these chambers, or even the necessity for them. When the surface drainage was over earth, gravel, and macadam roads, such as were common years ago in many suburban areas, there was a great quantity of road detritus carried to the disposal works and in many cases grit chambers were desirable if not necessary. Since then, in nearly every city, roads have been treated with asphalts or tars so that the road material carried away in the sewers is much less than before, and the disposal works is able to handle this without the necessity of a grit chamber.

However, with the development of the treatment of activated sludge it is evident that grit chambers are once more appearing in the picture.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—The attempt to secure a reasonably clean grit by the control of velocities in the grit chamber is a modern development. The detritus tank as generally found in England and exemplified in Germany at the Hamburg screening stations, and in Paris, France, at the Clichy pumping station, is not designed to attain this object, being practically a sedimentation tank having a brief detention period. In America the two deposit sewers, 16 ft. high by 8 ft. wide and ¼ mile long, built more than 40 years ago in connection with the main drainage works of Boston, Mass., afford an example of the same class. These are placed after the pumping station, their function being to prevent deposits in a long tunnel siphon under Dorchester Bay.

Modern German and American practice attempts to secure as complete a separation of the inorganic matter as practicable and the experiments carried out at Cleveland, Ohio, by Mr. Gascoigne have aided greatly in providing a scientific basis for design.

The experience at Cleveland with vanes separating the upper "flowing through" portions of the grit chamber from a lower compartment receiving the grit, was similar to that at the Dyckman Street screening plant in the Borough of Manhattan, New York City, where the resulting deposit contained a large quantity of organic matter.

An interesting attempt to maintain a fairly uniform velocity is illustrated in the design of the grit chamber of the Frankford Creek Intercepting Sewer, Philadelphia, Pa., where a pair of vertical vanes in the chamber are pivoted on axles at each side. The vanes are of such shape that when closed there remains a bell-shaped opening for the low flows. As the rates increase the leaves are opened until, at times of maximum flow, the entire cross-section of the chamber is available.

For large installations, where mechanical means for cleaning are justified, the use of hoppers or sumps appears to have advantages over the simpler and less expensive flat-bottomed type. In such installations the writer believes that cleaning by a grab-bucket is more reliable and, in the end, more economical than by the use of compressed air or by bucket elevators. It is hoped that others with a larger experience in this phase of sewage works operation will take this opportunity to furnish more specific information.

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

HARRISON P. EDDY,* M. AM. SOC. C. E.—The subject of grit chambers is one that has not been studied thoroughly in all cases. Sometimes, apparently, such chambers have been installed because the practice had been adopted elsewhere.

There have been many instances, however, where grit chambers have proved of value. At Worcester, Mass., about twenty years ago, they were installed as a means of preventing the large expenditure required for labor in the removal of heavy sand and gravel deposits from the sedimentation tanks. At that time the sewage of Worcester was treated by chemical precipitation.

At Fitchburg, Mass., where the topography of the city is very rugged and where there are many miles of gravel and cinder roads from which large quantities of sand and detritus reach the sewers, grit chambers have been installed at the up-stream end of a long inverted siphon leading to the treatment plant. It is of interest to note that in spite of this it has been necessary to clean the lower compartments of the Imhoff tanks at this plant. In these compartments accumulations of inert material consisting in part of sand, gravel, and cinders, gradually reduce the total effective capacity for sludge digestion until the material is removed. Designs have been prepared for an additional set of grit chambers, in order to reduce the frequency of cleaning the sludge compartments.

There has recently developed in the United States another problem which has rather overshadowed that of grit handling. This is the removal of oil. Oil comes from various places, such as automobile service stations, factories, and gas-works. At Akron, Ohio, as much as 300 gal. of heavy oil have been skimmed from the surface of existing Imhoff tanks in a single day, even if the plant was receiving only a moderate portion of the entire sewage flow. Similar conditions have occurred at Schenectady, N. Y., Milwaukee, Wis., Chicago, Ill., and Worcester; and, at Detroit, Mich., large quantities of oil flow from the sewers into the river.

In order to remove this oil promptly and inoffensively and at the same time remove the grit and coarser suspended solids, a skimming detritus tank has been devised and is now (1925) under construction at Akron in connection with its new sewage treatment plant. The detritus plant comprises two round tanks, 55 ft. in diameter and 13 ft. deep. The influent will be admitted to one side of the tanks and the effluent will pass out through submerged orifices. For average ultimate rates of flow these tanks will provide a detention of 15 min.

The tanks will be equipped with adjustable weirs and revolving arms, adjusted so as to skim the grease and cause it to flow on to small drying or absorbing beds. A part of the suspended solids, including the grit, will settle. Revolving plows will be provided to assist the passage of the settled solids to a central sump in each tank. These sumps will be connected with centrifugal pumps, which will draw the sludge and grit together with approximately 20% of the sewage from the bottom of the detritus tanks.

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

This concentrated sewage will flow through one of a pair of grit chambers. Each grit chamber will be about 3 ft. wide and have an effective depth of flow of 3.5 ft. Each chamber will be provided with a flight conveyor to scrape the grit from the chamber up an incline and discharge it into a storage chamber, from which it will be dropped into cars and thus carried to the fill.

The flow from the grit chambers will contain considerable suspended organic matter for the removal of the coarser portion of which two mechanical fine screens with $\frac{1}{8}$-in. slots, will be provided. These screens will be 6 ft. long by 6 ft. 6 in. in diameter. The screenings will be disposed of either by incineration or by use for filling.

The general plan of providing detritus tanks and accessory equipment has several advantages. Such an installation will prevent relatively large quantities of oil, grease, and floating solids from accumulating on the surface of the sewage in the Imhoff tanks and rendering them unsightly or expensive to maintain in a presentable condition. The detritus tanks will be at least as effective as grit chambers and fine screens in removing grit and the coarser suspended solids. This removal of the coarser suspended matter will reduce the danger of trouble with scum in the Imhoff tanks. In the case of Akron it is estimated that such detritus tanks for the entire sewage flow, with grit chambers and fine screens for the concentrated sewage equivalent to 20% of the total flow, will result in a substantial saving in the construction and operation costs over similar costs for grit chambers and fine screens for the total flow.

R. S. Rankin,* Esq. (by letter).—The design of the grit chamber for Decatur, Ill., possesses certain peculiarities uncommon to others, that may be of interest. The chamber is located at the end of a 60-in. intercepting sewer about 14 000 ft. long. Beyond the grit chamber are the Venturi meter, Imhoff tanks, and the remainder of the treatment works. Between the grit chamber and the Venturi meter, area and sufficient head have been provided for future sewage treatment developments. As a result of this provision there existed between the water levels in the grit chamber and in the Imhoff tanks, a difference in head, which varied for different quantities of sewage flow. The grit chamber design was based on the assumption that there would be a definite water level for any given flow through it with the water level at the outlet of the chamber lower by the amount which had been allowed for future devices. This difference in head is now controlled so as to produce a uniform velocity in the chamber.

In the design of the grit chamber, two major assumptions were made: (1) That the water level in the chamber would be the same as that in the incoming sewer; that is, the water level would rise and fall the same as in the sewer; and (2) that not more than two of the three "flowing through" compartments in the chamber would ever be used at one time.

The quantity and the level of the flow in the incoming sewer, which is 60 in. in diameter, varies according to the usual curve of a sewer having a

* With the Dorr Company, Inc., New York, N. Y.

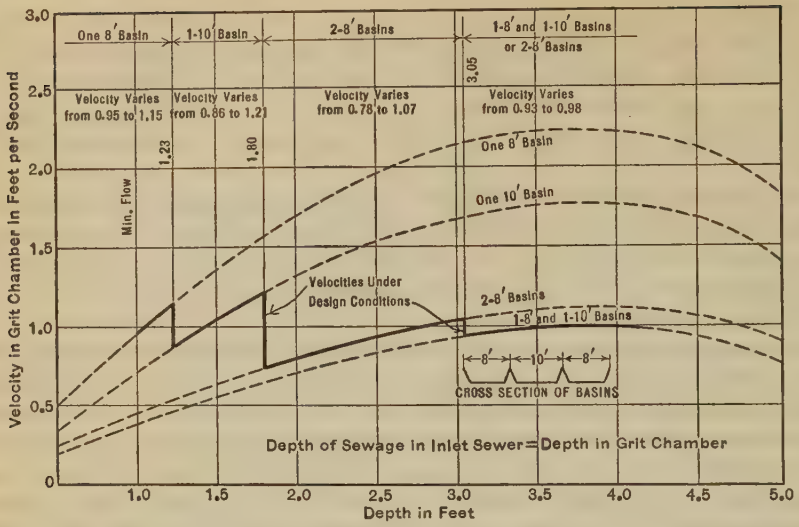


FIG. 9.—VELOCITIES IN GRIT CHAMBER, DECATUR, ILL.

circular cross-section. It was decided that an optimum velocity of 1 ft. per sec. would be desirable in the grit chamber. The theoretical cross-section that would produce this required velocity was computed for various levels. It was slightly modified to straighten out the irregularities to a section which would be readily constructed.

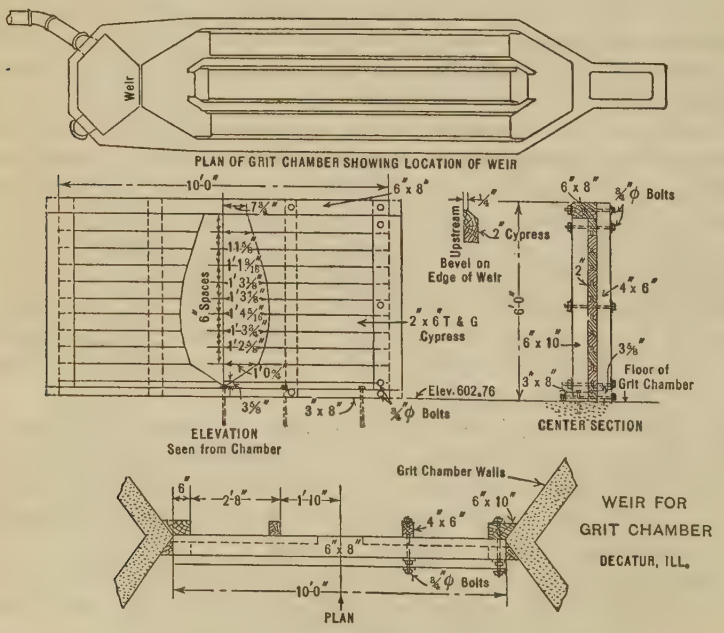


FIG. 10.

With the water level at any elevation corresponding to that in the incoming sewer, the theoretical flow and velocity in the grit chamber will be very close to 1 ft. per sec., as shown in Fig. 9.

In order to use up the head at the outlet and still maintain the correct level in the grit chamber, it was necessary to design a rather peculiar weir. This weir is of the submerged type designed on the basis that at any level there would be a definite head available for discharge. After several cut-and-try computations, a theoretical profile of the weir was determined and a section approaching this as nearly as practicable was adopted. The weir with this profile was constructed of wood and installed in the outlet as shown in Fig. 10, with the idea that modifications might have to be made. However, no modifications in the weir have become necessary, as current meter measurements in the grit chamber from time to time have shown that the velocities are very close to 1 ft. per sec.

The drainage of the grit chamber is accomplished by under-drains placed in a recess along the sides and near the bottom of each "flowing through" compartment, and consist of gravel-packed drainage pipes laid with open joints. The gravel is held in place by cast-iron gratings set flush with the side-wall of the chamber.

For removing the grit from the basin, each compartment has been provided with three outlets of 18-in. cast-iron pipe with a gate-valve control. The grit chamber is set high enough above the ground so that dump cars may run beneath on industrial tracks and the contents of any chamber may be discharged through these outlets directly into the cars.

The control of the flow is accomplished by means of wooden stop planks at either end and a swinging wooden gate at the inlet end. The operation of the grit chamber in general has worked out satisfactorily and conforms substantially with the design.

Arthur J. Martin,* Esq. (by letter).—There is little that the writer can usefully add to this discussion beyond saying how greatly he has been impressed by the thoroughness with which the subject has been studied in America.

The abandonment of macadam road surfaces, noted by Mr. Macallum, has been so general in Great Britain that in many towns there is little need for grit chambers unless the sewage is to be treated by the activated sludge process.

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Paper No. 1637

THE WATER SUPPLY SYSTEM OF MONTREAL, QUEBEC, CANADA*

By Frederick E. Field,† M. Am. Soc. C. E.

General

The water consumed by the City of Montreal is a mixture of the waters of the St. Lawrence River and the Ottawa River in varying proportions, depending on the stage of the rivers and the prevailing winds.

The St. Lawrence River water is rather hard, having an average alkalinity of about 95 parts per million and a permanent hardness of about 20 parts. The Ottawa River water is very soft, having an average alkalinity of about 10 parts per million and a very slight permanent hardness.

The population is served by two independent corporations, the City of Montreal and the Montreal Water and Power Company, a private company supplying water to several wards of the city and to some of the adjoining suburbs.

The Montreal Water and Power Company has its pumping and filtration plant on the river bank, at the foot of Charlevoix Street, with force mains leading to two reservoirs. The Outremont Reservoir has a capacity of 7 500 000 gal. with the water level at 470 city datum. During 1924 this Company had an average daily consumption of about 28 000 000 gal.

Municipal Works, Montreal, Que., Canada

Aqueduct Canal and Supply Conduit.—The City of Montreal obtains its water supply from the St. Lawrence River about two miles above the Lachine Rapids. Water from both a shore and a mid-stream intake is conveyed through an 8½-ft. concrete conduit, 5 miles long, to the site of the Filtration Works

* Presented before the Sanitary Engineering Division, Montreal, Que., Canada, October 15, 1925.

† Engr., Montreal Water Board, Montreal, West, Que., Canada.

and the Low-Level Pumping Station. As the capacity of this conduit is only about 90 000 000 gal. per day, the City during the last few years has completed the enlargement of the Montreal Aqueduct Canal to obtain sufficient water for present and future needs.

The entrance to this canal is an imposing structure comprising a concrete bridge and three head-gates, each 57 ft. wide and 18 ft. high. These gates are counterweighted and are operated by winches enclosed in towers. The canal, which is about 150 ft. wide and 15 ft. deep, follows the same general course as the supply conduit previously mentioned and terminates below the Filtration Works and Low-Level Pumping Station at the Old Wheel-House where there is sufficient head available for a limited power development.

Filtration Works.—The Filtration Works, designed for a nominal daily capacity of 50 000 000 gal. (Imperial), was first placed in service in 1918. The plant includes a low-lift pumping station; sixteen pre-filters, each having a net sand surface of 1 200 sq. ft.; sixteen final filters each having a net sand surface of about 16 000 sq. ft.; and two filtered-water basins, one having a capacity of 6 000 000 gal., and the other, recently constructed, having a capacity of 20 000 000 gal.

The pre-filters are cleaned by a reverse flow of air followed by a low-rate water wash, and the final filters are cleaned by washing machines which run on longitudinal tracks in each bay. These machines operate without lowering the water above the sand. They rake and wash the sand to any desired depth and leave the sand bed clean, uniform, and uncompacted.

The following tabulation shows the average characteristics of raw water for 1924:

Alkalinity	57	parts per million
Color	33	" " "
Turbidity	12	" " "
Temperature	48	" " "
Bacteria	5 020	per cu. cm.

Average Results of Operation for 1924.—The pre-filter effluent had a turbidity of 3 parts per million and contained 1 700 bacteria per cu. cm. The final filter effluent had a turbidity of 2 parts per million and contained 163 bacteria per cu. cm.

After sterilization by chlorine the water leaving the Fltration Works contained 6.5 bacteria per cu. cm. No chemicals other than the chlorine for the sterilization of the filtered water is used at this plant.

The percentage of wash water was as follows:

Pre-filters	0.39
Final filters	0.09
Total	0.48

The following tabulation gives the cost of operation per 1 000 000 gal., for 1924:

	Imperial.	United States.
Supervision	$0.17	$0.14
Pumping station	0.56	0.48
Pre-filters	0.28	0.23
Final filters	0.24	0.19
Laboratory	0.17	0.14
Sterilization	0.76	0.64
Electric current	1.53	1.27
Building and grounds	0.07	0.06
Total	$3.78	$3.15

New Low-Level Pumping Station.—This station is located directly adjoining the wall of the new 20 000 000-gal. filtered-water basin from which the pumps obtain their supply. The pump-room is arranged for six motor-driven centrifugal pumps having a unit capacity of 30 000 000 gal. per day, against a total head of 250 ft.

The electrical rooms contain transformers, switching gear, and other equipment required for receiving the 10 500-volt alternating current and stepping it down to 2 200 volts for the main pumping units and to lower voltages for other uses. The building also includes a machine shop, several store rooms, and the offices of the Montreal Water Board.

This station is arranged for future extension to include six additional 30 000 000-gal. per day pumping units and their necessary appurtenances. The pumps discharge through 36-in. pipe into 72-in. riveted steel headers, each of which in an adjoining building, known as the Distribution Control Building, divide into two 48-in. force mains which become the main arteries of the City's distribution system.

Lachine Canal Crossings.—The extension of the 48-in. force mains from the Low-Level Pumping Station to the city is difficult and expensive because of the necessity of crossing under the Lachine Canal.

The crossing at Wellington Street includes two reinforced concrete shafts, 18 ft. in diameter and about 105 ft. deep, so that the concrete connecting tunnel which is about 300 ft. long is in solid rock. This crossing is designed to include one 48-in. and two 36-in. force mains which throughout the crossing are of riveted steel pipe. On the north side of this tunnel, the 48-in. cast-iron force main is being extended toward the east side of the city.

City Reservoirs and Booster Pumping Stations.—The City of Montreal has no large storage reservoirs, but is generally supplied by direct pumping, and the several reservoirs serve as pressure equalizers rather than as storage reservoirs.

The largest of these reservoirs is the McTavish Reservoir, holding 37 000 000 gal. This reservoir controls the head on the pumps at the Low-Level Pumping

Station. The Peel Street Reservoir on the mountain holds 1 750 000 gal. This reservoir receives its supply from a booster pumping station at McTavish Reservoir.

The highest reservoir in the city is the Cedar Avenue Reservoir which holds 200 000 gal. This reservoir receives its supply from the Peel Street Reservoir by means of a booster pumping station on Cedar Avenue.

In the west end of the city there are three tanks holding 100 000 gal., which are supplied by a booster station at Madison Avenue. In the east end of the city the Papineau Pumping Station serves to increase the water pressure in this district.

Water Consumption.—During 1924 the daily average output of the municipal plant was 71 600 000 gal. (Imperial), which includes 58 200 000 gal. of filtered water and 13 400 000 gal. of sterilized raw water. The per capita consumption is between 135 and 145 gal. per day. The water supply is not metered except for manufacturing plants, laundries, etc.

Present-Day Situations.—The completion of the Montreal Aqueduct Canal has brought to the site of the Filtration Works and the Low-Level Pumping Station a supply of water adequate for present and future needs. When the new Low-Level Pumping Station has its six 30 000 000-gal. per day pumps in operation, the pumping capacity will be adequate for some years and with its possible extension to house six additional 30 000 000-gal. pumps future needs are safeguarded.

The laying of the first of the six 48-in. force mains is well advanced and when this is connected up the distribution system will be much improved. The remaining force mains will be extended as required.

The Filtration Works designed for 50 000 000 gal. per day (Imperial), is delivering about 60 000 000 gal. and still is not adequate for present needs. Funds have been made available for extending the filtration work, and the preliminary studies are well advanced. These studies contemplate coagulation basins and rapid sand filters. The water supply will be obtained from the Montreal Aqueduct Canal and will flow by gravity through the plant to the existing filtered-water basins.

Construction Department.—In 1920, the City authorized the organization of the Montreal Water Board which has since made investigations, prepared plans and specifications, and supervised the construction of all extensions and improvements of the water supply system of the City of Montreal.

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Paper No. 1638

A CENTURY AND A HALF OF AMERICAN ENGINEERING

ADDRESS AT THE ANNUAL CONVENTION IN PHILADELPHIA, PA., OCTOBER 5, 1926.

By George S. Davison,* President, Am. Soc. C. E.

A great nation is viewing retrospectively its life of a century and a half, and taking an inventory of its achievements. The main factors in its growth lie in its area, population, and wealth. The Continental United States of to-day covers ten times the number of square miles within the borders of the original thirteen States. Its population has increased from about 3 000 000 to 115 000 000. No means are at hand for estimating the wealth of the nation on its natal day, but this much is certain—it was insignificant in comparison with the $400 000 000 000 that stand to its credit to-day.

It is eminently proper that members of the American Society of Civil Engineers as citizens of such a nation should gather here in Philadelphia, where its birth is being celebrated, to attest their interest in its affairs and their loyalty to its aims and purposes. It is fitting that while here members of a Society the life of which has covered half that of the United States, should inquire what part the Civil Engineer has played in the creation of those conditions which, within one hundred and fifty years, have transformed America from a few struggling colonies into a leading nation of the world. We are using the term "civil engineer" in its broadest sense, in order to include all those who through scientific processes are directing the great sources of power in Nature that they may be most useful to mankind. We recognize that the mechanical engineer, the mining engineer, the electrical engineer, the hydraulic engineer, the sanitary engineer, or any engineer of special calling, is only practicing such a branch of civil engineering as his title would indicate and his professional knowledge justify.

This does not mean that to the engineer alone belongs all the credit for the colossal advances we now enjoy as a people. His efforts have been involved

* Pres., Gulf Refining Co., Pittsburgh, Pa.

with those of the artisan, the industrial manager, the inventor, the capitalist, the statesman, and many others; but it is his ability "to design as well as to direct engineering works" that must be relied on to bring successful results not only out of his own vision but out of the vision of others, and to cause the dreams of man to come true.

The inhabitants of that part of America which comprised the original thirteen States were mainly of English, Dutch, German, and French descent, whose forebears had left their native lands either in search of adventure or to escape conditions that were intolerable. Even had they possessed the knowledge of the arts and sciences as of their day and of the place from whence they came, their new surroundings offered no encouragement to make use of such knowledge. They had anticipated the simple life mingled with hardships; and they were not disappointed. Their existence resembled camp life, with little idea when they would move and where next they would go. Temporary expediency answered every purpose. Labor-saving devices, even had they been conceived, could not have been enjoyed because of a lack of capital with which to develop and install them. Their ambitions were confined to securing for themselves food, shelter, and clothing. To till the soil, to fell timber, and to weave the cloth were their main tasks. Industry had developed in only a few directions. It might be remarked that one of these was the production of iron, which was smelted with charcoal as fuel.

The territory occupied lay along the sea and extended less than 100 miles toward the Appalachian Range. Commerce sought the sea. Inland transportation either by land or water was inconsiderable.

The manufacture of power as an essential for rail transportation and industry had not yet been initiated. In fact, at that early time, the untamed wind, the fall of waters, and the beast of burden were the only supplements to manual labor. Material advancement from this condition was long delayed. It was not until 1830 that the steam railroad came, and even twenty years later that great agent of civilization had scarcely begun to play its part. Turning, then, to the object of our inquiry, namely, what has the civil engineer accomplished, we shall find his greatest triumphs within the past seventy-five years.

The early pursuit of the people was agriculture. Land was the first consideration. Its area was to be determined and apportioned. Therefore, land surveying early engaged the attention of the engineer. It was in 1824 that the first civil engineering school in America was opened, twenty years after the establishment of the United States Military Academy. The founder of this school, Stephen Van Rensselaer, a man of affairs, served as a member of the Erie Canal Commission from its inception in 1816 until his death in 1839. His contact with the problems of that time had given him a vision of the influence that the technical mind would have over the future affairs of the people. In stating his object in founding this institution, he said:

"I have established a school for the purpose of instructing persons who may choose to apply themselves in the application of science to the common purpose of life."

The first class in civil engineering was graduated in 1835.

These facts show what little was being done one hundred years ago to educate a young man for the profession; from which it may be deduced that the demands then made on the profession for service were few in number and simple in character. The story of the country's progress in applied science in its first half century confirms this view.

At that time transportation was—as it has continued to be—the great problem underlying the prosperity of the nation. Then it dealt with sailing vessels and horse-drawn vehicles. Later, the highway yielded to the canal, then the canal to the railway, and now the railway is viewing with alarm the return of the highway to public favor.

The beast of burden has all but disappeared, and human energy has been replaced with mechanical power from Nature's great storehouse. That public service may be brought to the home, the switch, the valve, and the spigot have been installed. That worthless land may be redeemed, irrigation and drainage have been effected. That the worker may be comfortably sheltered and profitably employed, the skyscraper and the industrial plant have been built. That commerce may move whither it will, the face of Nature has been smoothed with bridge and tunnel.

Relation of Various Engineering Fields

Passing now from the general to the particular, it may be instructive to present a brief historical review of the more important pursuits in which the civil engineer has specialized. To deal with the many subjects in anything like a chronological order is difficult, as the various branches of civil engineering mingle indiscriminately both as to time of development and scope of work. Such, for instance, is Surveying, the first topic treated; certainly it was one of the earliest forms of engineering in America and it still has a large and important following among engineers. Surveying is, so to speak, a common factor in all engineering work.

In order of development the agencies of transportation come next, namely, Waterways, Highways, and Railways, in order. Logically, the study of Power succeeds these for it is a *sine qua non* for all transportation. Following hard on the development of transportation came a variety of attendant engineering pursuits, as Ship Building, Bridge Building, City Planning, and Tunneling.

With the growth of the city a number of vital needs presented themselves. Especially did the community look to the engineer for the protection of health, and of him was demanded the development of Water Supply, and of Sewerage and Sewage Disposal. Likewise, with the concentration of urban population, the need of housing and industrial development inspired the great field of Structural Engineering. In due season, agriculture like industry, demanded expansion, economy, and conservation; thus as one of the later, indeed almost the latest, great National engineering development came Irrigation.

In this order, more or less logical, will be treated the more important events in the various engineering fields during the last one hundred and fifty years of American history, each topic under its own appropriate heading.

Surveying

The art of surveying, like other processes of engineering, has made important progress during the last century and a half. It is indeed a far cry from the rude compass and chain used by the Father of His Country to the delicate aerial cameras and submarine sounding devices in use at the present day.

An epochal event in the development of surveying occurred toward the end of the Eighteenth Century. Ramsden, an English instrument maker, perfected a dividing engine with which he produced a theodolite having a circle 36 in. in diameter and capable of being read to a single second of arc.

The Nineteenth Century has been notable for the introduction of the steel tape to take the place of the old surveyor's chain, the use of better and more precise methods for various kinds of surveying work, particularly geodetic surveying, and the development of special instruments.

Even as late as 1887, the chain was spoken of as the usual instrument utilized in land surveying and the steel tape, said to have first been made from hoop skirt wire, was considered only a means of accurate measurement. It was an expensive device. Modern manufacturing methods, together with its superior accuracy, have made it the standard instrument.

In geodetic surveying, bars of various materials, two in number, used alternately with careful alignment, were employed to measure primary base lines. It was a very expensive and not a highly accurate method. More recently, the invar tape with a coefficient of expansion roughly one-thirtieth that of steel, has made base-line measurements, with an accuracy of 1 in 1 000 000, possible under almost any temperature conditions.

As indicating the extreme accuracy and ingenuity of modern surveying devices for special purposes the latest aneroid barometer which exhibits readable variations for a difference of a few inches in elevation may be mentioned. Perhaps the greatest skill applied to surveying methods at present is being given to aerial surveying. Wonderfully accurate cameras have been developed. Some of the devices for reducing the photographs to engineering maps are even more remarkable, including a stereoscopic measuring table, permitting contours for areas of high relief to be drawn on the photograph to an interval even as small as 10 ft.

A system for surveying public lands was authorized as early as 1784; it has been since modified, to define the locations of principal meridians, standard parallels, township lines, and section lines. This work is in charge of the General Land Office.

The United States Coast Survey, although authorized in 1807, was not instituted until 1817, while material progress was not made until about 1832. Commencing on the Atlantic Coast, the work eventually included the Gulf of Mexico and the Pacific Coast. About 1870, authority was given to connect the surveys of the two coasts. These labors involved various geodetic determinations which gave an added significance to the work of the Survey, so that in 1878 it became officially the "Coast and Geodetic Survey" under which name it now operates.

Somewhat akin to this Government Bureau may be mentioned the United States Geological Survey, which, however, restricts its activities to the inland territory of the United States. The Geological Survey is the outgrowth of several separate activities under various Government departments which were merged in 1879. Besides other duties the Geological Survey makes topographic surveys and maps, geological maps, and hydrographic surveys of the inland waters of the United States.*

One of the most recent developments for taking soundings through water is called the Sonic Depth Finder. This instrument, attached to a vessel, catches the sounds reflected from the bottom of the ocean and by delicate measuring devices enables the depths through which the sound has passed to be measured.

Of modern surveying methods that from the air bids fair to outstrip all others for reconnaissance work, for map revision, and for securing the flat map features in extensive topographic work. A series of photographs taken from an aeroplane flying at a known height is made inclusive enough to cover the entire area with sufficient overlapping. A ground survey covering objects and elevations capable of identification on the air photographs links the picture to the actual ground topography. This method is in frequent use and under rapid development.

Little would the surveyor of one hundred and fifty years ago have dreamed that such a rapid and reasonably accurate method were possible. Thus has modern engineering science succeeded in utilizing all the vast improvements in methods to make the profession still further a benefit to humanity.

Waterways

Prior to the founding of our Government and for some time thereafter no effort was made to improve for navigation purposes the inland waterways nor the inlets and harbors bordering on the sea coast. In the course of events the Federal Government assumed control of all natural watercourses. In recent years, with the expenditure of monies taken out of the general funds of the nation, it has deepened and otherwise improved the channels leading into deep-sea ports, and the more important rivers.

A long time prior to these improvements, however, private capital took up the question of developing the natural resources and expanding the trade of the country by means of artificially constructed waterways. The earlier plans with a few exceptions were confined to reaching near-by markets with the products of mine and factory. More ambitious projects than these, involving extensive mileage, were necessary for general trading purposes.

The first long-distance canal was the Erie Canal completed in 1825. This, together with the Hudson River, connected the Great Lakes with the Harbor of New York and assisted materially in extending trade with the seaboard into Northern Ohio and a considerable area contiguous to the Great Lakes. Once this territory was reached, at least three important canals were built from points on Lake Erie to the Ohio River, thus providing an all-water route from the Hudson and Delaware Rivers to the Mississippi Valley. This circuitous way was necessary to get an all-water route around the Appalachian Range.

* *Transactions*, Am. Soc. C. E. (1905), Vol. LIV, Pt. B, p. 419.

Meanwhile, the State of Pennsylvania developed a route across the Allegheny Mountains, thus shortening the distance from tide-water to the Ohio River by one-half. This artery of travel was a combination of railway and canal, with inclined planes between Hollidaysburg and Johnstown, by means of which the loaded canal-boats, when separated into sections, could be carried over the mountains.

Canal building extended through a period of about thirty-five years, or until 1850. It has been estimated that in the era of artificial waterways 4 500 miles of canals were built at a cost of $214 000 000. Much of this mileage was doomed to financial failure before it was finished, because meanwhile railroads had been inaugurated in the United States (1830). Once their advantages were realized the death knell of those canals serving the interior was sounded, so that even now the channels and locks of the greater portion of these canals have been entirely obliterated.

The only important canal that has survived the competition of the railroads is the Erie Canal. It is State-owned and has undergone a complete reconstruction to accommodate boats of larger tonnage and deeper draft, to be operated by steam. Even with this modification the freight carried is inconsiderable in comparison with the traffic on the railroads paralleling it, its tonnage for 1924 being equal to about 60 000 carloads.

However, even since the advent of railroads there have been examples of the canalization of important streams. Some of these earlier projects were initiated with private capital, but, because of the policy of the Government during the past thirty years to own and operate such works, these ventures have been taken over through condemnation proceedings. Under the general program of serving navigation, the canalization of rivers is continuing under the direction of the War Department. In the case of some more important rivers, such as the Mississippi, the improvements are under the jurisdiction of Commissions, organized under Federal acts.

A most important part of the program of the Government in the interest of navigation deals with the improvement of the harbors along the coast and on the Great Lakes for vessels of deep draft. Since 1902, this work and that relating to rivers has been carried on under the direction of the U. S. Army Engineers, as provided in the River and Harbor Act. The enormous work accomplished since the passage of this Act in 1902 may be realized when it is understood that for these purposes fully $500 000 000 has been spent.

Great as may appear the sum spent by the Government in the interest of navigation, it is far overshadowed by the enormous amounts expended by local harbor authorities and private interests to provide docks and equipments to handle the traffic at terminals. The cost of the grand total of all governmental and private construction in the interest of waterways runs into billions of dollars, the greater part of which has been accomplished within the past fifty years.

Beyond all this special mention should be made of the Panama Canal, constructed within the years 1907-14 and costing $367 000 000.

In addition to carrying out this vast construction program, to the engineer is committed the responsibility of maintaining these great works so that trade and commerce may be served to the highest degree of effectiveness.

Highways

Like Topsy, the earlier roads in America "just grew". As communities became active centers of business and as local governments began to function, inexperienced supervision assumed the responsibility of maintaining these highways, to which only such attention was given as would keep them in a barely passable condition. In the course of time companies were organized for the purpose of assuming charge of the more important routes, and they charged toll for the service rendered. Meanwhile, canals had demonstrated their advantages over roads, by furnishing better service at a lower cost; but soon both fell into disfavor on the advent of the steam railroad.

The encouragement of a government liberal with its subsidies to those who would construct railroads in virgin territory, and the prospects of a rich return on traffic already created by the former methods of transportation, made investments in the new field so attractive that capital for building turnpikes "dried up". Prior to the fall of the highway before the irresistible advance of the railroad, the Conestoga wagon had been the freight car, the stage served as a passenger coach, and the Pony Express as the fast mail.

The highway, however, ceased to be an empire builder and the turnpike companies settled down to bide their time hoping for a better day to dawn. In due course the delay was rewarded, in most cases by condemnation proceedings, through which the public compensated the stockholders of such companies.

Early road building afforded little opportunity to the engineer. It was only at a very recent date that his services became necessary in highway construction. Forty years after he had taken over the building of railroads, the machine which at first had the appearance of a plaything became a demonstrator of the advantages of good roads. I refer to the bicycle. By 1890, it had millions of devotees, who, through the League of American Wheelmen, demanded better roads of the State and local authorities.

Although this gesture made a lasting impression, the bicycle did not enjoy its popularity long enough to secure prompt results. Better arguments for good roads came with the motor car. In 1900, motor vehicles to the number of 8 000 were registered in the United States; in 1910 there were 468 000; and in 1925 there were about 20 000 000. While the operation of motor cars was at first confined to city streets, in due course of time the desire to use the country roads could not be suppressed and the main highways were improved to meet the demand of the motorist.

The internal combustion engine has wrought a miracle and the automobile has settled down for a long visit. What at first was a luxury has become a necessity. Even the farmer, the clerk, and the laborer must have his pleasure car. Auto-buses have increased travel over important highways and built up new business over routes that were almost abandoned. Commerce finds the massive truck suited to its purpose in the transportation of freight. The

railway that once so ruthlessly robbed the highway of its good name, is now seeking mercy at the hands of "good roads".

To meet the advancing popular needs dustless roads were developed—usually by surface treatment of the existing macadam. In due time bituminous macadam and bituminous concrete were evolved and contemporaneously the cement concrete surface. However, with all these improvements, highway engineering has had the utmost difficulty in keeping abreast of the increasing traffic demands.

There are 3 000 000 miles of roads in the United States. Of this mileage, at the end of 1924, much had been improved—of low type construction, 308 000 miles; of macadam, 115 000 miles; and of high type roads, 46 500 miles, a total of 468 500 miles. On this mileage there had been spent about $1 000 000 000. In 1924 alone 31 540 miles of roads were improved. The civil engineer may view the situation with complacency, as the highway will continue to require his services for some years to come. His specifications of five years ago for road building did not anticipate the heavy wheel loads and congested traffic that must now be met. To-day, he is using better material, deeper foundations, more scientific drainage, and heavier and more enduring bridges. What was only recently "good practice" soon had to be renewed.

While the engineer has been building this great network of highways, he has been designing the cars that will use them and, in turn, finding the fuel to run the cars.

Indeed, modernizing the idea in an old nursery rhyme, it may be said:

"This is the road that Jack built,
This is the car of Jack's design,
That sped over the road that Jack built,

"This is the gas that Jack produced,
That was used in the car of Jack's design,
That sped over the road that Jack built."

The great strides that are being made in road building, and in the manufacture of vehicles to travel on them, is strictly due to the civil engineer, and it is limited only by the amount of money which the public is willing to spend in their improvements. However, it must not be overlooked that this great task on which the engineer is engaged has already rendered useless millions of the capital which he spent for other clients in former years. The buses *de luxe* have thrown the interurban line on the scrap heap, and have struck a serious blow at the steam railroad with respect to short-haul of passengers and freight.

Railways

Industrial railways—horse-drawn—preceded the steam railroad in America, and that as early as 1806. The steam railroad came into existence in 1830 with a length for that year of 30 miles. The existing system of 250 000 miles is the country's greatest asset and its present state of efficiency constitutes the proudest achievement of the engineer.

The fact that the weight on the driving wheels of the locomotive has increased steadily from less than 20 tons in its earlier years to nearly 400 tons is the answer to the question of what single movement has had the greatest influence on our prosperity. These heavier locomotives have been followed by cars of greater capacity and heavier rails of improved quality. Roadbed and structures have been re-designed to meet the greater stresses imposed on them. Main tracks have been paralleled and terminals enlarged.

The handling of longer trains and an increased number of them by these more powerful engines had been made possible by the introduction of the air-brake, the automatic coupler, the automatic safety signal, and, now, of train control. The safety, speed, and comfort of passenger travel have been advanced through these same improvements and in addition by the introduction of the palace car, of the vestibule platform, of steam heating and electric lighting of coaches, and of stations of monumental design and great utility. Coincident with these betterments, grade crossings have been abolished, grades and curvatures reduced, alignments improved, locomotive fuel efficiencies greatly increased, and electric traction installed where it was necessary or desirable.

It is a matter of record that in the olden days of horse-drawn vehicles the cost of moving freight over improved turnpikes was about 20 cents per ton-mile, and the rate per ton-mile on canals was not less than 3 cents. For at least twenty years before the general advance to higher costs occasioned by the World War, the average revenue per ton-mile on all railroads in the United States was less than 0.75 cent. The increase in advantages to the traveling public over the days of the stage coach has been in speed and comfort rather than through reduction of rates of fare, although fares have undergone a revision downward.

Granite blocks were first used as a foundation, on which the rails were laid either directly or with intermediate stringers. Soon, however, the present method of supporting the rail on cross-ties was adopted.

The first rails were of wood, protected on the wearing surface by iron straps. Iron rails came into use as early as 1835. They were of the general form of the T-rail of to-day. The first ones, imported from England, were in 15-ft. lengths, weighing 36 lb. to the yard. The first iron rails rolled in America appeared in 1845, at which time the weight had increased to 56 lb. and the length to 30 ft. Steel rails were imported to a small extent about 1860. American steel rails were produced by the Bessemer process as early as 1865, and by 1880 most of the original iron rails had been replaced by steel, the maximum weight being 67 lb. The present general form of splice-bar joints has been used for at least fifty years. Meanwhile, the weight of the rail has advanced to a maximum of 130 lb. and open-hearth steel has gained favor over the Bessemer product.

The first locomotives were wood burners from actual use. Coal was used as early as 1855. The railroads penetrating the Southwestern and Pacific Coast territories have been using oil for fuel for twenty-five years. Within that same period, the electric locomotive has been adopted for handling trains in and out of certain busy terminals, particularly where smoke from steam

locomotives would not be permissible. Some studies have been made looking to a more extended adoption of this form of power, and we can look forward to many important developments in this practice. Following the idea that where the necessary water power is available the generation and distribution of hydro-electric power may show greater economies and adaptability to the service required than steam power, about 450 miles of one great railroad have been electrified.

The passenger coach of the early days of railroading would accommodate a mere dozen passengers. There were as many patterns of coaches as there were railroads, but by 1840 the coach had assumed its present form, mounted on center-pin trucks. (The railway truck, incidentally, is an American invention.) The length of the body was 40 ft., the width 8 ft., and the capacity about 20 passengers. The early coaches were heated by stoves, at first burning wood, and, later, coal. At night their interiors were lighted with candles. On his rounds the conductor found his way about the dim car with the aid of a hand lantern. The coaches were of wood, and frequently in cases of wreck they caught fire from the heating systems, cremating passengers who might otherwise have escaped.

The coach continued to increase in size to the present length of 80 ft. and width of 10 ft., in weight to 122 000 lb., and in seating capacity to 84 passengers. The stoves have been superseded by steam heat from the locomotive. The method of lighting has passed through the stages of kerosene lamps, gas (stored under pressure in tanks beneath the floor), and, finally, electric current, produced at first by a steam engine located in the baggage compartment and more recently by generators belted to the axle of the coach. The water cooler in the end of the coach is another American idea, which deserves honorable mention because of a modest and faithful career that has not been properly appreciated.

Efforts to furnish sleeping quarters for passengers were made as early as 1836, but the real Pullman did not make its appearance until 1860; and, in 1867, there were just forty-eight of them in service. The first diners were called "hotel cars," and consisted of a kitchen installed in the end of a sleeping car. Although the present form of the dining car followed shortly, the call of "twenty minutes for dinner" was still heard throughout the land until about twenty-five years ago.

The air-brake was first applied to a passenger train in 1868, just preceding the introduction of automatic couplers. It has undergone marked improvement both in principle and effectiveness since its original installation.

The first railroad freight car had a capacity of only a few tons. Its development as to running gear, couplers, automatic brakes, and increase in dimensions is similar to that of the passenger coach. To-day, its maximum carrying capacity is 70 tons. At first, freight cars were loaded and unloaded by hand. Many improvements have been made by which such labor has been greatly reduced. The grab bucket and the "hopper bottom" are examples. An entire car, loaded with coal, may be run into a cage-like framework that is revolved transversely to discharge the contents of the car.

The stage and the canal-boat moved at the rate of 4 miles per hour. The limited trains of to-day carry the passenger, the letter, and the express package at a speed of 50 miles per hour. The economies in time and money resulting from the higher speed are not susceptible of calculation.

The horse car for urban service was a novelty seventy years ago. A few decades later the cable car was evolved as an improvement. It was believed that besides increasing the capacity of surface lines and providing quicker service, great economies could be effected by a mechanically operated system with the power developed at a central station and distributed by a moving cable. The cable car, however, was quickly abandoned for the trolley car except on streets of steep grade where it is still in use because of its high degree of safety. When electric-current transmission by feed wire with application through motors on the car was introduced, it sprang into favor quickly, and a whirlwind construction program was followed until fifteen years ago. This movement wiped out the horse car, and, in many cases, seriously affected the local passenger business of the steam railroad.

Fifty years ago, the congestion of vehicular traffic on city streets suggested the idea that part of it must be transferred elsewhere. Elevated lines were constructed on which multiple-car trains hauled by dirty, noisy, and jerky steam locomotives were operated. While this development enhanced property in the vicinity of the stations, the damage resulting to property abutting on such elevated lines was so enormous that they failed of general adoption. A quarter of a centry ago the subway idea was advanced as a better method of mass transportation for the reason that it serves the public equally well and does not have the same blighting effect on adjacent properties.

Power

To describe power development in the past one hundred and fifty years is in effect to review it from the age of the guilds. During that time revolutionary strides have been made and such undreamed-of applications have proved successful that there is scarcely one of the devices in use a century and a half ago that has not become obsolete.

The first use of steam, which has been the principal prime mover of the Nation's industries, was the foundation of man's present freedom from toil—the beginning of our present mode of living. One hundred and fifty-two years ago, James Watt, the mathematical instrument maker of Glasgow, perfected the reciprocating steam engine. Oliver Evans, the American stepfather of the steam engine, and the earliest of the New World's great engineers, first advocated the high-pressure steam engine.

For nearly a century the steam engine was the undisputed source of mechanical power in America. With respect to the operation of factories, it abolished the inflexible water-wheel and windmill. The steam engine became known in time to create a system of transportation theretofore practically unknown—the railroad; it raised the floating commerce of the seas above the hanging helplessness of sails.

A climax in the building of steam engines was marked by the one installed to supply power for the Centennial Exposition held in this city (Philadelphia)

in 1876. This gigantic vertical engine weighing 700 tons, a Corliss machine of 1 400 h.p., was, in its day, the last word in American steam engineering.

At this same Exposition was displayed a then novel form of energy—electricity. It depended on steam or water power for generation, but, once made, could be transmitted easily by wires—no belts, gears, or rope drives were needed. The prime mover could thereby enlarge the field through which its energy might be distributed.

One hundred years after Oliver Evans' first compound engine, the steam turbine appeared. Its high speed proved to be ideally adapted to the generation of electricity. Later, the Pelton water-wheel, an impulse turbine, was connected with the dynamo and thereby long neglected water-power sites were harnessed and their power spread by electricity to industrial centers. Electricity is now so easily transmitted, so simple of control, and so powerful in work, that two-thirds of the Nation's machinery is directly operated by electrical power.

Another progressive step in the flexible distribution of power has been made through the use of the air compressor.

The past twenty-five years has witnessed a wonderful expansion in the building and use of the internal combustion engine. It has found its principal field in transportation. By automobile, motor-boat, and aeroplane, the passenger may be his own engineer and pilot as he journeys by land, by water, or through the air.

A simple method of measuring the advance of the Nation in recent years is to note that in 1900 it generated 13 000 000 industrial h.p. and that in 1925 the quantity had soared to 55 000 000 h.p. In 1900, it utilized 2 500 000 000 kw-hr. of electricity; in 1925, this had been increased to 68 000 000 000 kw-hr.

The development of power in the United States is one of the marvels of all times. Its wide use conserves the individual's energies and shortens his working hours, gives him higher wages through quantity production, supplies him with many articles of living never before available for general distribution, and, in short, raises his standard of living above that of rulers of other centuries.

Ship Building

Ship building in America is as old as Jamestown and older than Plymouth. Every Colony had its shipyards. Prior to the Revolution the annual launchings had reached 27 000 tons—Gloucester fishing schooners, deep-sea whalers, and coastwise traders made up the Colonial fleet. Commerce with the outside world was handled generally in foreign bottoms.

In 1781 a merchant ship, the *Alfred*, was acquired by the United States Government and was constituted the first (and only) ship of the Navy. John Paul Jones served as First Lieutenant on this vessel. The *Alfred* was of 220 tons displacement; to-day, the size of the Navy is 2 253 000 displacement tons. As compared with the *Alfred*, the battleship, *California*, is of 33 190 tons displacement.

In 1791, Alexander Hamilton, Secretary of the Navy, reported an American Merchant Marine of 476 274 tons. The Merchant Marine of to-day comprises 14 878 761 tons.

The *Clermont,* a vessel of 160 tons built by Robert Fulton in 1807, was the first steam vessel. Fulton also, in 1814, built the first war vessel to be propelled by steam—a ship of about 1 200 tons. The first steam vessel to cross the Atlantic was the *Savannah* (of 350 tons) which sailed on her maiden voyage in May, 1819, from the Southern port for which she was named. In 1843, the first iron ship in America was built.

The *Great Western,* probably the first English steamship designed especially for the trans-Atlantic trade, created a sensation when she arrived in America on her maiden voyage in 1838. She was of 1 340 tons and required 15 days to make this ocean passage. As an evidence of progress, it should be stated that there is now being built an American vessel of 22 000 tons, the speed of which will be such that she can make the trans-Atlantic trip in 7 days.

Fulton, in building steam vessels, used paddle-wheels about 15 ft. in diameter, operated directly by the connecting rod of the engine. The walking-beam method of transmitting this power is just one hundred years old, and still maintains its popularity for inland navigation. The screw propeller, invented by John Ericsson, was first applied in the United States to the steamer, *Princeton,* in 1843. It is now the accepted method of propulsion for vessels, especially on the high seas.

The change from coal to oil for boiler fuel in large merchant and naval ships began in 1900. American practice in handling the steam from boilers to its point of application successively passed through the stages of the simple engine of Fulton, and the compound engine of seventy years ago, to the triple-expansion engine, now about fifty years in vogue. Twenty years ago the direct-drive steam turbine began to find preference over the reciprocating steam engine, for certain types of vessels. One of the latest developments with respect to power for ships is the internal combustion engine using oil for fuel; and the very latest device for the application of marine power is the electric drive, with a choice of steam turbine or Diesel engine as the prime mover.

Admittedly, the United States is not in the first rank in the matter of a merchant marine, although she is fully up to standard in naval equipment. She is well supplied with tonnage plying coastwise and on the Great Lakes. Since the days of Robert Fulton her inland waterways have been routes for transportation. The difficulty in maintaining uninterrupted traffic on some of these streams and the competition of the railroads, which were better and quicker, caused the abandonment of much of this service. However, while these waterways have been passing through periods of uselessness, Congress, with the advice of the War Department, has been providing funds for revetment and canalization on them. The engineer has been engaged for years on such works, and now, with improved channels on the one hand and a greatly increased cost of rail transportation on the other, inland water transportation is again coming to the fore as one of the important assets of the nation.

Bridge Building

The art of bridge building is believed to be as old as civilization, but the science of it could not have antedated the first demonstration of the law of gravitation. Empirical rules and judgment based on experience were the guides in early construction.

The masonry arch is one of the oldest forms of the bridge. Besides serving a useful purpose, it afforded the early engineer a means of expressing his artistic taste. Primitive America was forced to sacrifice the beauty and durability of the arch for designs of less costly material. The first stone arch in this country was built in 1800, and, although devoted to public use, its cost came from a private purse. Beginning with 1820, famous aqueducts to carry water supplies to several of the larger cities were constructed. With the advent of the railroad, a limited number of arches were included in its original construction. Later, many large railroad bridges were renewed in stone.

Until seventy-five years ago the question of cost determined to what extent the masonry arch should be used. Then dawned an era when communities with funds available to meet their asthetic desires returned to the ancient idea of giving utility an artistic finish. A large number of these structures may be found in American cities, the combined work of the engineer and architect.

Continuing to follow both the useful and the esthetic in the arch bridge, cement concrete began to take the place of stone about thirty years ago, and in a short time stone was used only in the voussoirs. Later, the introduction of steel reinforcement reduced the weight and cost of arch bridges without impairing their strength, so that the use of stone is now limited almost exclusively to decoration.

The earlier bridges in America were of wood. Prior to 1776 only a few had been built, their lengths comprising a succession of short spans (about 20 ft.). It is recorded that in 1792 a span of 175 ft. was constructed. Before the steam railroad became a factor, many bridges of that length, and even greater, were built by such designers as Timothy Palmer, Theodore Burr, and Louis Wernwag. In the longer spans, combinations of the arch and truss were used, and much ingenuity in design was displayed, the framing timbers being determined from a study of models and from experience gained by the failures of previous attempts.

An epoch in bridge engineering occurred in 1840 when William Howe patented the truss which has since borne his name. In it iron rods were used for vertical tension members. This simplified the connections at the joints and made the determination of stresses in members easy.

The late Squire Whipple, Hon. M. Am. Soc. C. E., took out letters patent in 1841 for his bowstring truss, in which cast iron was used for compression members and wrought iron for tension members. From that time iron bridges were much in favor and many types of trusses were developed and built, some of which were identified by the names of the engineers who first suggested them.

The scientific computation of stresses in bridge members was demonstrated in 1847. Without regard to the material used in articulated bridge structures,

serious difficulties had presented themselves in the design and construction of the connections between the members. The Howe truss had eliminated this difficulty as to wooden structures. In metal structures, and particularly those of cast and wrought iron in combination, the solution of the trouble was much delayed. The pin-connected truss that was evolved about 1858, principally to meet the requirements for long spans involving difficult problems in erection, simplified the connections. After this form of truss had gained great favor there followed the famous "pin *versus* rivet" war, between American and English engineers, which lasted for many years. In recent years shop and field facilities have so improved in workmanship and cost in this country that American practice has drifted definitely toward riveted construction.

The relative cheapness of cast iron made it the popular material for compression members for many years. Cast iron in highway bridge construction was long in vogue for architectural effects after its use in main members had been abandoned. The largest, all cast-iron, bridge in this country was built over the Schuylkill River, at Chestnut Street, in this city (Philadelphia) in 1864. It is still in use.

The failure of a cast-iron railroad bridge in 1876, accompanied by a serious loss of life, resulted in the discarding of this material in such bridges. A similar action followed shortly with respect to highway bridges. Construction in wrought iron throughout was the prevailing practice until open-hearth steel became available at a price and of a quality that made it more desirable than iron. However, the substitution of steel for iron had been increasing gradually before the wholesale adoption of soft steel. The first all-steel truss bridge in this country was built over the Missouri River, at Glasgow, Mo., in 1879. Crucible steel was used in the arched spans of the Eads Bridge, at St. Louis, Mo., which structure was finished about 1875.

Many problems arising out of the intersections of highways and railroads with navigable waters have been presented to the bridge engineer. Government responsibility over waterways demands that the channels be protected by the provision of large clearances. The suspension bridge, the cantilever, or the steel arch, is used where the approaches permit an elevation of the channel span that will provide the proper vertical clearance. When sufficient clearance under a fixed span cannot be secured within a reasonable combined cost for construction and damages, movable spans on a lower grade line must be used. In such cases a combination of the science of the structural engineer and the ingenuity of the mechanical mind is necessary. Up to a certain period the horizontally revolving drawn span, balanced on a center pier, was believed to fit any case. Spans that may be moved vertically are improvements on that idea; the popularity of these types is illustrated by the great number and variety now in use.

From the first iron truss bridge of 77 ft., built in 1840, the maximum length of spans has increased until in this year of 1926 the longest simple truss is 720 ft.

The first suspension bridge was built in 1801. It was a highway bridge, with a span of 70 ft. Chains were used as suspension members, on which was

hung the wooden floor. In July, 1926, in this city (Philadelphia) there was dedicated a suspension span of 1 750 ft., the cables being of the usual built-up wire section.

The first cantilever bridge of importance was built in 1876 with a center span of 375 ft. The longest cantilever in the United States at this time is 1 182 ft.

The first important metal arch bridge was the Eads Bridge. Its longest span, 520 ft., was completed in 1874. The Hell Gate Arch Bridge, built in recent years (1912-17), is 1 000 ft. long.

City Planning

One hundred and fifty years ago there were only three cities in this country with any considerable population. Boston had no systematic layout; New York and Philadelphia were being developed on a rectangular basis. The New York plan was the result of the work of the first City Plan Commission in the United States, created in 1807; the basis was a mason's wire screen. It, however, was influenced by an earlier idea from Philadelphia. The Philadelphia design was based on the Penn plan of 1682, comprising a width of about eight blocks, extending north and south within the area between the Delaware and Schuylkill Rivers.

In a recently compiled "History of Pennsylvania", I have discovered what is believed to be an account against William Penn for the work done on this plan. The items are as follows:

"For taking courses and the soundings of the Delaware, seven weeks with Captain Markham, £10.

"To victuals and drink put on board the shallop at different times, £3.

"To my attendance at first commission William Hague, Nat Allen, and John Beazor, no charge."

(Even then, it seems, engineers were prone to give public advice free of charge.)

"To my taking courses of Schuylkill, etc., for sounding and placing Philadelphia upon Schuylkill, etc., £6.

"To lodging Captain Markham and William Hague in my house with diet and liquor for treats, £7.

"Total, including liquors, £26."

In 1791 there was produced, for a small section of what is now the capital of this country, a plan patterned to some extent after the Paris idea of the Seventeenth and Eighteenth Centuries as further developed by Baron Haussmann, under Napoleon III.

It is asserted that the early conception of the rectangular plan became an imprint on the municipal landscape of the United States, due largely to the authorized system of Government land surveys, adopted in 1785 at the suggestion of Thomas Jefferson. Whatever may have been the actuating cause, it is true that, with few exceptions until recent times, the plans of American cities have followed this line of least resistance as far as surveying is concerned. Consideration of topography played but little part.

One notable exception is that of Buffalo, N. Y., laid out by Joseph Ellicott, the younger brother of one Andrew Ellicott, who was the first Surveyor-

General of the United States and who completed the plan of the City of Washington on the basis of the designs of Major L'Enfant. Detroit, Mich., in 1805, had a plan fashioned somewhat on the L'Enfant idea of diagonal thoroughfares. With these exceptions, until the time of the European and English Garden City movement, there was little variety in the planning of American cities. Kansas City, Mo., presents a case in which imposition of the rectangular plan has recently been greatly modified and delicately improved by the intelligent use of boulevards, parkways, and parks, and which has stimulated in the outskirts residential subdivisions of a variety and attractive character fitting the topography.

Another notable exception, artistic in its conception at the time, was the layout of the Centennial Exposition held in Philadelphia fifty years ago, followed by the more modern conception of Burnham for the World's Fair in Chicago, Ill., in 1902; and, later, at Buffalo, St. Louis, and San Francisco, Calif. These artistic and attractive developments made their impression on the work of the architect, the builder, and the engineer in the layout of municipalities and subdivisions. Notable examples were brought forward at the time of the World War, during which many towns and houses for workers were built after the highest ideals of usefulness, economy, and attractiveness of layout.

The fundamental basis of good city planning is the groundwork of the city engineer. If he is true to his calling he is always a city planner in embryo, because he must think ahead in planning public works and utilities. His understanding and appreciation of many of the elements of its attractiveness and his contribution to efficiency and economy have received much impetus during the past quarter of a century and it is only reasonable that special groups of engineers should organize themselves as city planners.

City planning has become an important issue in many large cities. It is to be regretted that its practice has been so long delayed. The recent problem of traffic relief has thrust itself suddenly on the engineer planner, who has been dismayed at the appalling cost of the reconstruction necessary to meet present-day traffic conditions.

Tunneling

Tunnels are built in the interest of transportation. The first tunnel in the United States, completed in 1821, was a part of a canal. The second, finished shortly afterward, extended the operations of a coal mine. Tunnels have been used widely for purposes of water supply and even sanitation; but their extensive use is found in connection with railroads. Just as the bridge may "level up" lines across a valley, so the tunnel can "level down" lines at summits.

In recent years highway traffic has become of such economic importance that the vehicular tunnel has laid claim to the services of the engineer. Just as it has always been necessary to carry railways and highways by bridges over waterways to preserve the rights of navigation, so, in these later years, the tunnel has been brought into service to relieve the streets of such traffic as may well be handled underground. It is within the past twenty-five years

only that the engineer has been giving attention to the subaqueous tunnel that may provide facilities not readily supplied by a ferry or bridge.

Tunneling has been a relatively expensive operation. This nation began its existence one hundred and fifty years ago with much hope and courage, but with a lean purse. Even the need for transportation had to be created. When the necessity arrived, it had to be met with plans that felt the pinch of economy, for the Appalachian Range loomed as a barrier against any extension of territory to the west. This obstacle could for a time be avoided by way of the Great Lakes, which idea led to an all-water route across New York State to Lake Erie and from Lake Erie to the Ohio River.

The next plan that found favor was the shortening of this long journey to the Middle West by way of the Pennsylvania Canal and the incline planes of the Allegheny Portage Railroad. The heads of the inclines on the two sides of the mountain were connected through the first railroad tunnel constructed on this Continent, which was completed in 1833. Through this tunnel the eastbound traffic of the Pennsylvania Railroad still passes. Since the completion of this first tunnel the Appalachian Range has held no terrors for the engineer. He has pierced its sides and foot-hills many times in the interest of the railroads.

An epoch in the history of American tunnels was the completion, fifty years ago, of the Hoosac Tunnel, which extends for 4¾ miles under the Hoosac Mountains in Western Massachusetts. This enterprise met with many discouraging delays during the twenty years between its commencement and its completion. Since that experience many railroad tunnels have been built of different lengths, with the Moffat Tunnel in Colorado, 6½ miles long, as the most ambitious example. As a rule, all such tunnels were driven through rock. There came a day, however, when the engineer was compelled to work through soft ground above open water, and, later, to drive subaqueous tunnels through all classes of material.

One of the earliest of the important subaqueous tunnels is the pair of tubes for the Hudson and Manhattan Railroad (New York, N. Y.). It outranks the Hoosac Tunnel as to discouragements and delays, there being an interval of twenty-six years between its start and its finish. The difficulties with these two noted tunnels were sources of great instruction to the engineer. The lessons there learned have made the later problems in tunnel work extremely simple.

The so-called tunnels for subways under city streets have been constructed, for the most part, in open cut, the finished form of the subway being rectangular, of structural steel, encased in concrete. The skill of the engineer in this class of work is directed mainly to maintaining traffic at the street surface, protecting the foundations of existing structures at the building line, and permitting all sub-surface utilities to be operated continuously.

There are certain inventions or practices that have introduced epochs in tunnel driving, and that have combined to bring the art to its present degree of perfection. The rock drill was an American invention of 1849. The compressed air drill, in connection with high-grade explosives, spelled success for the Hoosac Tunnel in the early Seventies. The air-lock made the use of

compressed air in subaqueous work possible. The adoption of the shield method for driving tunnels, first used in 1869, has eliminated the hazards of driving through soft materials. In its first use it was driven forward by hydraulic jacks, but, nowadays, compressed air under which tunnel work is usually carried on, more often is used as the driving force.

Water Supply

The first water-works plant in the United States was built in 1652. The supply was by gravity from springs. Pumping machinery was used for the first time in 1754. The pump was of *Lignum vitæ,* and the pipe to the reservoir was of hemlock logs. In 1762 this pump was replaced by others made of iron.

Steam was first used for pumping in 1800. The engines were constructed largely of wood, even the boiler being partly of this material. Wooden pipes were used entirely, and not until 1804 were cast-iron pipes introduced.

In 1800 the total number of water-works plants in the United States was 16, all, of course, in the larger cities. In the next fifty years this number increased to 83. In 1875 there were 422; by 1900, there were about 3 500.

By 1866, American water-works engineers had begun to take an interest in foreign practice with respect to filtration. The first slow sand filter plant in America was built in 1871, but it was not until 1880 that any considerable attention was paid in the United States to the purification of water supplies. Fanning's "Treatise on Hydraulic and Water Supply Engineering" published in 1884 (the first American work on the subject) contained only nine pages descriptive of filtering processes. By 1900, however, 20 cities in the United States were supplied with water that had been purified by the English or slow sand method of filtration, while 149 cities and towns were using mechanical filters, many being of the pressure type. Soon after 1900, concrete began to replace wood and steel tanks for mechanical filters and the majority of such filters built since that time have been of the gravity type. Recent practice favors the mechanical type and many of the existing slow sand filter plants are being rebuilt with "pre-filters" of that type.

To-day, more than 10 000 cities and towns in the United States have public water supplies. This tremendous growth has been the result of the requirements both for supplies of pure water for domestic and other uses and for fire protection. It has been made possible by the advent of cast-iron pipe, the invention of improved pumping machinery, and the development of advanced methods of purification. One of the most noticeable changes in the art has been that which has taken place in pumping machinery within the last fifteen or twenty years. Increased economy had been attained by the design of great direct-acting, compound, and triple-expansion pumping engines. Within recent years, however, motor-driven turbine or centrifugal pumps have replaced the direct-acting engines on account of their lower first cost and economy in operation.

Another development of the last thirty years has been the quite general introduction of the meter system. With the adoption of meters has come a change in the method of charging for water from the former flat, or fixture,

rate to schedules based on the quantity of water used. This method reduces waste, promotes economies in operation, and tends to maintain expenditures for the pumping and filtering plants at a minimum.

Sewerage and Sewage Disposal

The development of sewerage systems and methods of sewage disposal has been coincident with the growth of cities.

The first application of engineering skill to the design of sewers was about 1857-58. Empirical formulas, however, were used as a basis of design until, in 1889, the rational method was advocated. The combined system of sewers was in general use up to 1880 when the first systems of separate sewers were constructed.

The earliest sewage pumping stations were put into service in 1884 and 1885. As the discharge of raw sewage into the streams increased, their condition became intolerable. In 1872, investigations relative to stream pollution were begun by the State Board of Health of Massachusetts. In 1887, the Lawrence Experiment Station of the Massachusetts State Board of Health was established. The researches of this Station in connection with the fundamental biological conditions underlying processes of sewage treatment constitute an unparalleled contribution to the art of sewage disposal.

Among the first methods of sewage treatment in the United States were attempts at sub-surface irrigation and broad irrigation. Broad irrigation was soon replaced, particularly in New England, by intermittent sand filtration. Chemical precipitation followed in the latter Eighties. Contact beds for the oxidation of organic solids in sewage were introduced into America during the next decade. Trickling filters have largely superseded contact beds. The first municipal filter of this type was put into service in 1908, and the fine screening of sewage was introduced into this country in the same year.

The earliest American installation of Imhoff tanks was made in 1911, and the first attempt at separate sludge digestion on a large scale, shortly afterward.

The most recent development of importance in sewage treatment has been in the activated sludge process, which is the outcome of experiments on the aeration of sewage by numerous American workers, particularly at the Lawrence Experiment Station in 1912. The first activated sludge plant was built in this country in 1916, and several large plants have been constructed since that date.

Disinfection of sewage was practised as early as 1892 by the use of chlorine produced electrolytically. The efficiency of hypochlorite of lime was demonstrated later. At present, liquid chlorine is more widely used.

Structural Engineering

One-half of the one hundred and fifty years with which we are dealing had passed before metal was used as a building material. In this earlier period outside and intermediate walls and interior rows of wooden posts bearing longitudinal timbers, all supporting the floor joists, carried the weight of the

building and its contents to the foundation walls, which were of the continuous type. With this character of construction, buildings of more than five stories were seldom built, because such structures were not readily accessible in their upper floors, and, in addition, required walls and foundations of unreasonable thicknesses, thus adding greatly to the dead weight of the building. The increased fire and wind hazards were also important factors.

About 1850 cast iron came into use in the form of columns and beams, and later it was used very extensively in the fronts of business houses for ornamental effect. A large five-story building covering a city block in New York with its four faces of cast iron, built in 1859, is still in use. The framework of the Crystal Palace, located on the present site of Bryant Park, New York, and built in 1853, was entirely of cast and wrought iron.

Iron beams were first rolled in 1854. It was probably as late as 1870, or about fifty-five years ago, that beams, channels, angles, and tees were available. About 1884 the rolling of shapes changed from iron to steel, and within ten years but little iron was rolled into structural shapes.

With the advent of the rolled beam efforts were made to reduce the risks from fire in large buildings by using these shapes as floor-beams between which were sprung brick segmental arches. This practice prevailed generally until the Chicago fire in 1871, during which incident the disastrous effects of fire on the beams was demonstrated. Thereafter resort was had to other practices that involved various forms of hollow tile, and when arches between metal floor-beams were of this material, the beams themselves were encased in hollow tile forms. Intermediate arches of light weight, such as of cinder concrete, also have been popular. Of recent years the reinforced concrete slab floor has found great favor.

There came a time when it was recognized that land areas must be used more intensively in the business sections of cities. In the vertical expansion of buildings to meet this necessity the important problem was to provide easy access to the higher floors. It was sixty years ago that the first suspended type of elevator was installed. The first elevators were steam-operated, the suspending cable being wound around a drum. The hydraulic plunger type followed. Now, the electric elevator has largely superseded the earlier types.

Thanks to the introduction of structural shapes and the elevator, buildings began to creep skyward. By 1885, ten-story buildings were being built. In 1887, skeleton construction was established by its use in all but party walls. In 1891, or just thirty-five years ago, the use of weight-bearing walls for high buildings ceased, and the full skeleton type of construction had been adopted. At that time a twenty-story height was attained. The practice of supporting the walls of each floor independently of the others and upon the skeleton structure permitted the discarding of continuous foundations and the adoption of isolated footings as supports for the posts.

Buildings of forty stories in height are becoming quite common, and there are a few examples of even fifty stories. The United States was somewhat behind England in the use of metal columns and beams in building construction, but, once the wrought-iron shapes were available for this purpose, the

American engineer moved swiftly to the skyscraper, which has proved to be a most wonderful structure, in that it conserves space horizontally, provides as nearly as possible fire and storm-proof quarters, and affords better air and natural light, while its floor spaces are adjustable to any combination of partitions. From the first rolled beam to the all-skeleton building was but thirty-seven years.

Within the past twenty years great advances have been made in the quality and quantity of Portland cement. The use of this material in connection with reinforcing bars (a new shape in steel) in casting beams and posts in place has brought reinforced concrete into competition with steel in skeleton constructed buildings.

Irrigation

Probably no phase of the Engineering Profession is more tinged with romance than the work of the irrigation engineer. The creation of fertile fields, where once the sage and cactus suffered a shriveled existence, lends itself too readily to fanciful dreams to escape glorification by authors and scenario writers.

Irrigation dates back several thousands of years before the time of Christ, when the fields of Egypt, once desert lands, were transformed into garden spots by primeval engineers who constructed hydraulic works to control the periodic overflow of the Nile for its deposit of rich alluvial mud. Its ancient and honorable status is also attested by the fact that practically the first Scriptural reference to agriculture relates to irrigation in the Valley of the Euphrates, for in Genesis, 2:10, we find this statement:

"And a river went out of Eden to water the garden; and from thence it was parted, and became into four heads."

Irrigation continues in practice in the Far East, and, although it is still, to some extent, using devices similar to those employed centuries ago, some of the world's major works, and some of the most advanced practices in irrigation, are to be found in India and Egypt.

Excepting only the primitive efforts of the Indians along the Southwest border, irrigation was not practiced on any scale in the United States until 1847. The plains of the Middle West were so readily accessible for agriculture that it was unnecessary to resort to the more expensive development of farm irrigation. The hopelessness of attempting agriculture by ordinary methods, in that region of restricted rainfall which they then occupied, led the Mormons to initiate irrigation. This movement has since been developed to such an extent that there are to-day in the United States more than 63 000 individual irrigation enterprises. Early irrigation works, while simple when compared with more recent practice, demonstrated the possibilities of using the once worthless desert land for raising crops.

Irrigation was limited to the smaller and less expensive developments until 1902, when the plan was inaugurated of using Government funds, derived from the sale of public lands, for the construction of large projects, the funds thus expended to be repaid to the Government by the settlers on the land reclaimed.

This work has been prosecuted under the direction of the United States Reclamation Service, and the great dams and reservoirs built by this organization are outstanding monuments to modern engineering skill. To date, this agency has expended about $165 000 000 on construction works alone and has reclaimed nearly 2 000 000 acres of land. The cropped area on Federal projects for the year 1925 was 1 216 610 acres, and the yield that year was valued at $66 488 500, or $54.65 per acre.

The Eleventh Census of the United States, compiled in 1889, shows 3 564 416 acres as being irrigated. In the following ten years, this figure was more than doubled to a total of 7 263 813 acres in 1899. In 1919, there were 231 541 irrigated farms with a total of 19 191 716 acres being irrigated, this being nearly 2% of all the farm land, and 3.8% of the improved farm area, of the United States. The total capital invested in irrigation enterprises in the United States, as reported by the last Federal Census (1920), amounted to $697 657 328, and the total value of irrigated crops for the census year approximated $800 000 000.

The economic value of irrigation in respect to Western agricultural and industrial expansion, and the important relationship that this expansion bears to National development and security, in both its economic and military aspects, can hardly be over-estimated.

The Civil Engineer deals with quantity, time, cause, and effect. His problems involve dimensions, form, and motion. He creates to-day and destroys to-morrow, but out of the destruction new forms arise of improved design and greater utility. Experience may be a guide for his future policy, but he cannot rest on repetitions of methods that may have brought him fame. He must be alert to the ever changing conditions about him, recognize the problems they create, and apply wisdom and ingenuity to their solution.

He accepts physical laws as immutable. In the past one hundred and fifty years he has developed no new ones, but he has subjected the familiar ones to a more searching analysis. New forms of fuel have been found; the steam engine has been developed; electricity born; and the key to distribution of power found. Less drudgery, better health, and more leisure—these are the realities of American life to-day to which the Civil Engineer has contributed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1639

VENTURI TUBE CHARACTERISTICS*

By J. W. Ledoux,† M. Am. Soc. C. E.

With Discussion by Messrs. J. E. Gibson, D. E. Davis, Joseph A. Russell, W. S. Pardoe, Bayard F. Snow, Morrough P. O'Brien, Arthur L. Collins, and J. W. Ledoux.

Synopsis

This paper discusses the Venturi tube and gives the coefficients of discharge and losses of head determined from a series of actual tests. made under the direction of the writer, of nineteen tubes ranging in size from 30 in. to 1 in. There is also given in the paper results of tests of eight tubes, obtained by other experimenters.

The effect of various angles of convergence and divergence, tube ratios, and size of tubes on the coefficients of discharge and losses of head, is discussed; and the conclusion is reached that for Venturi tubes ranging in size from 4 to 30 in., the coefficient of discharge is approximately the same, and ranges from 0.96 to 0.985, with an average of about 0.977 for high velocities, and from 0.90 to 0.94 for low velocities.

From the results of the tests of the twenty-four tubes, the following formula for loss of head is deduced:

$$f = \frac{V_1^{1.96}}{403}$$

which is roughly expressed as:

$$f = \frac{V_1^2}{470}$$

which is stated in another way as,

$$f = 0.13\, H \left(\frac{R^4}{R^4 - 1}\right)$$

* Published in November, 1926, *Proceedings*.
† Cons. Engr., Philadelphia, Pa.

The opinion is also given that the Venturi tube is only surpassed in accuracy by actual weighing or volumetric measurements. While a V-notch or rectangular weir is an excellent device for approximate measurements, it is not sufficiently accurate for use as a basis for checking the accuracy of a well-designed Venturi tube.

Every hydraulic engineer is familiar with the main characteristics of the flow of water through orifices and Venturi tubes. The advantage of the Venturi tube over an orifice is that for an equal differential head the unrecoverable loss of head for the orifice is much greater than with the Venturi tube.

It was formerly believed that Venturi tubes, to be accurate, should be made with definite and invariable lines; that the ratio of the main diameter to that of the throat should be fixed, and that the angles of divergence and convergence should always be the same. Within the past ten years, exhaustive tests have been made, which show that such requirements are unnecessary, and that the ratios as well as the angles of divergence may vary through a wide range without materially affecting the coefficient of discharge.

The most convenient formula for the Venturi tube is as follows:

$$V = c \left(\frac{2\, g\, H}{R^4 - 1} \right)^{\frac{1}{2}}$$

in which,

V = velocity through the main, or D section, in feet per second.
H = the Venturi head, in feet of water.
R = the ratio of main diameter, D, to throat diameter, d.
g = the acceleration of gravity.
c = the coefficient of discharge through the Venturi tube.

The unrecoverable loss of head through a clean Venturi tube may be expressed approximately by the formula:

$$f = \frac{V_1^2}{470}$$

in which,

f = the total loss of head across the Venturi tube, measured in feet.
V_1 = the velocity through the throat, in feet per second.

The exponent of V_1 is designated n in subsequent references. For,

$$c = 0.977,$$

and since,

$$V^2 = \frac{V_1^2}{R^4}$$

$$f = 0.13\, H \left(\frac{R^4}{R^4 - 1} \right)$$

It must be understood that these approximate formulas will not apply invariably to all tubes, as the tendency is toward larger values of f for smaller throat diameters.

It was also thought at one time that for a given velocity through the throat the loss of head was greater for the higher ratio of diameters. Actual tests do not confirm this with certainty. As to the angle of divergence of the downstream end of the Venturi tube, in many tubes this is 2½°; in many others, it is 6½°, but the effect on the coefficient is imperceptible.

With a given velocity through the throat it cannot be stated with certainty that the loss of head, f, is proportional or inversely proportional to this angle; neither does the loss of head vary exactly as the square of the velocity through the throat. The average of all the results available, including the writer's and those of other experimenters, shows the exponent, n, to be 1.961, the average formula being:

$$f = \frac{V_1^{1.961}}{403}$$

It was formerly taken for granted that coefficients of Venturi tubes varied with the size of the tube, the higher coefficients being obtained with the larger diameters. While that is probably true for tubes more than 4 in. in diameter, as compared with those less than 2 in. in diameter, there does not seem to be a great difference between the tubes as small as 4 in. in diameter and those as great as 24 in., or greater. William S. Pardoe, M. Am. Soc. C. E., has developed a formula which indicates that coefficients are higher with large tubes than with small ones, but the results of tests do not establish conclusively the invariable reliability of such a formula.

In making tests for coefficients of Venturi tubes, it is necessary to measure the quantity of water with great accuracy, and as the Venturi tube is such an accurate device itself, it cannot be checked with any less accurate apparatus. The only devices that are more accurate are tanks and weighing machines. All the tests herein referred to for coefficients, on which conclusions are based, were made on the weighing apparatus at the University of Pennsylvania, or the tank apparatus at Cornell University. For loss of head, however, the data obtained by weirs are near enough to warrant retaining the results, but for coefficients, weirs are not sufficiently accurate.

The measurement of the Venturi head also requires the greatest accuracy. Formerly, these heads were measured by manometers, but it was found that a much more reliable plan was to use hook-gauges, determining the actual levels to which the water rises in the main section of the tube and in the throat section, and taking the difference between those levels as the Venturi head. Such measurements of head are correct to 0.02 in., whereas with manometers, errors of 0.1 in. are common and almost unavoidable.

It was once believed that the coefficients of discharge were greater with low than with high throat velocities, but this opinion has been disproved by overwhelming evidence, and every tube tested under the writer's direction has shown the coefficient to increase rapidly with the increase of throat velocity from zero up to 8 or 10 ft. per sec.

TABLE 1.—FORMER TESTS FOR LOSS OF HEAD.

Test number.	Ratio of diameters.	Diameter of main, in inches.	Diameter of throat, in inches.	Angle of convergence, in degrees.	Angle of divergence, in degrees.	Losses of Head, f, in Feet, for Various Throat Velocities.							Values of Constants in Formula for Loss of Head, $f = \frac{V_1^n}{a}$.	
						10 ft.	15 ft.	20 ft.	25 ft.	30 ft.	35 ft.	40 ft.	a.	n.
1	3	108	36	10½	21½	0.19	0.39	0.70	1.08	1.53	2.13*	2.70*	477	1.94
2	2.11	54	25.5	10½	21½	0.23	0.53	0.94	1.43*	2.10*	2.85*	3.77	453	2.02
3	2.13	48	22.5	10½	21½	0.16	0.34	0.62	1.00	1.45*	1.98*	2.60*	698	2.03
4	3	48	16	10½	21½	0.15	0.34	0.62	0.98	1.43	1.95*	2.60*	804	2.07
5	3	12	4	10½	21½	0.26	0.56	0.97	1.45	2.10	2.80*	3.65*	303	1.90
6	2.33	12	5.16	10½	21½	0.25	0.55	0.94	1.47	2.10	2.68*	3.65	355	1.94
7	4.27	1.625	0.38	10½	21½	0.29	0.89	1.50	2.10	2.80	3.50	4.40	340	1.99

NOTE.—Test No. 1 made by Herschel, at Holyoke Flume, *Transactions*, Am. Soc. C. E., Vol. XVII (1887), p. 228.
" " 2 " " Marx, Wing and Hoskins, *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 471.
" " 3 " " Herschel, using weir.
" " 4 " " " " "
" " 5 " " " " tank.
" " 6 " at Cornell University, using tank.
" " 7 " by Coker and Strickland, *Transactions*, Eng. Inst. of Canada, 1902; also, "Hydraulics," by Hughes and Safford, 1912, p. 118.

*Loss of head figures interpolated on logarithmic paper by the writer.

TABLE 2.—LEDOUX TESTS FOR VALUES OF c, f, a, AND n.

Test number.	Ratio of diameter.	Diameter of main, in inches.	Diameter of throat, in inches.	Angle of convergence, in degrees.	Angle of divergence, in degrees.	Factors.	Values of Coefficient, c, and Lost Head, f, in Feet, for Various Throat Velocities. 1 ft.	2 ft.	4 ft.	6 ft.	8 ft.	10 ft.	15 ft.	20 ft.	25 ft.	30 ft.	35 ft.	40 ft.	Values of Constants in Formula for Loss of Head, $f = \frac{V_1^n}{a}$. a.	n.
8	1.5	24	16	10½	6½	c	0.930	0.952	0.966	0.973	0.976	0.977	0.977	0.976	0.975	0.973	0.970	0.969	242	1.67
						f	.....	.....	.....	.....	.....	0.20	0.38	0.63	0.92	1.25	1.62	1.97*	....	....
9	1.67	20	12	10½	6½	c	.....	0.968	0.977	0.979	0.980	0.980	0.981	0.982	0.982	0.982	.....	.....	846	2.14
						f	.....	.....	.....	.....	.....	0.16	0.40	0.73	1.16	1.75	2.40	3.15*	....	....
10	2	16	8	10½	2½	c	0.935	0.947	0.960	0.968	0.971	0.972	.....	.....	.....	.....	.....	.....	....	....
11	2	12	6	10½	6½	c	0.912	0.949	0.965	0.970	0.973	0.974	0.975	0.976	0.976	0.975	.....	.....	353	1.86
						f	.....	.....	.....	.....	.....	0.20	0.44	0.77	1.14	1.52	2.10*	2.70*	....	....
12	1.47	6.02	4.01	10½	6½	c	0.951	0.964	0.976	0.985	0.990	0.991	0.989	0.989	0.989	0.989	0.989	.....	636	2.03
						f	.....	.....	.....	.....	.....	0.19	0.39	0.71	1.11	1.57	2.16	2.85*	....	....
13	2.08	7.95	3.8	10½	2½	c	0.930	0.970	0.978	0.981	0.981	0.981	0.982	0.984	0.987	0.989	0.990	.....	....	....
14	1.995	6.04	3.028	10½	6½	c	0.922	0.942	0.956	0.966	0.973	0.975	0.975	0.9 5	0.975	0.975	.....	.....	237	1.87
						f	.....	.....	.....	.....	.....	0.32	0.65	1.13	1.76	2.42	3.04	4.12	....	....
15	2	6	3	10½	2½	c	0.930	0.950	0.968	0.973	0.975	0.975	0.976	0.976	0.976	0.976	.....	.....	388	1.93
						f	.....	.....	.....	.....	.....	0.22	0.48	0.83	1.30	1.82	2.47*	3.20*	....	....
16	2.24	6.08	2.72	10½	6½	c	0.910	0.931	0.950	0.960	0.966	0.969	0.969	0.969	0.969	0.969	.....	.....	....	....
						f	.....	.....	.....	.....	.....	0.40	1.0	1.85	2.90	4.10	5.55	.....	....	....
17	3	8	2.67	10½	2½	c	0.930	0.960	0.969	0.974	0.977	0.978	0.981	0.982	0.982	0.982	.....	.....	603	2.08
						f	.....	.....	.....	.....	.....	0.20	0.45	0.87	1.33	1.87	2.77	3.58*	....	....
18	2.98	6.02	2.02	10½	6½	c	0.895	0.908	0.923	0.933	0.942	0.946	0.947	0.946	0.945	0.945	0.945	0.945	235	1.98
						f	.....	.....	.....	.....	.....	0.40	0.90	1.60	2.50	3.50	4.77	6.20*	....	....
19	1.50	3	2	10½	6½	c	.....	.....	.....	.....	.....	0.963	0.966	0.965	0.962	0.962	9.66	.....	248	1.79
						f	.....	.....	.....	.....	.....	0.25	0.46	0.77	1.31	1.77	2.34	3.00*	....	....
20	1.99	3.97	2	10½	2½	c	0.930	0.945	0.956	0.963	0.966	0.968	0.970	0.970	0.970	0.970	0.970	.....	306	1.92
						f	.....	.....	.....	.....	.....	0.27	0.59	1.03	1.55	2.20	2.97	3.85†	....	....
21	1.97	3.928	2	10½	2½	c	0.925	0.941	0.955	0.963	0.968	0.972	0.973	0.975	0.976	0.976	.....	.....	....	....
22	2.61	4	1.532	10½	6½	c	0.900	0.922	0.945	0.954	0.959	0.962	0.966	0.969	0.970	.....	.....	.....	310	2.07
						f	.....	.....	.....	.....	.....	0.40	0.88	1.53	2.55	3.75†	5.10†	6.70†	....	....
23	2.01	2.01	1	10½	2½	c	0.928	0.936	0.945	0.951	0.956	0.960	0.963	0.965	0.966	.....	.....	.....	181	1.75
						f	.....	.....	.....	.....	.....	0.28	0.60	1.10	1.52	1.97	2.70†	3.45†	....	....
24	2	1	0.5	10½	6½	c	0.900	0.912	0.932	0.942	0.947	0.951	0.957	0.960	.....	.....	.....	.....	....	....
25	1.53	0.625	0.409	10½	6½	c	0.880	0.901	0.932	0.946	0.953	0.955	0.959	0.960	.....	.....	.....	.....	....	....
26	Averages of all the coefficients, (Tests Nos. 8 to 25).					c	0.919	0.941	0.957	0.964	0.968	0.969	.....	0.971	0.973	0.974	.....	.....	....	....
27	" omitting Tests Nos. 24 and 25.....					c	0.923	0.946	0.960	0.967	0.971	0.971	.....	0.973	0.973	0.974	.....	.....	....	....
28	" of Tests Nos. 8, 9, 10, and 11.....					c	0.926	0.954	0.968	0.973	0.975	0.976	.....	0.978	0.978	0.977	.....	.....	....	....
29	" of Tests Nos. 8, 9, and 10.....					c	0.932	0.956	0.968	0.974	0.976	0.976	.....	0.979	0.978	0.978	.....	.....	....	....
30	" Tests Nos. 8 and 9.....					c	0.930	0.960	0.972	0.976	0.978	0.978	.....	0.979	0.978	0.977	.....	.....	....	....

*Results interpolated.

†Loss of head figures interpolated on logarithmic paper by the writer.

To show the findings from various previous tests for loss of head, Table 1 has been compiled. The results of all the tests, except No. 7, followed an exponential formula; that is, they conform almost exactly to a straight line on logarithmic paper. The Coker and Strickland results, however, showed a considerable variation in the middle part of the series.

Opposite each test are given values of the constants, a and n, in the formula for the loss of head, in the form:

$$f = \frac{V^n}{a}$$

To deduce the values of the constants, a and n, the actual losses of head and throat velocities were plotted on logarithmic paper, and an equation was written for the straight line passing through the points representing loss of head.

For coefficients, only those results were used wherein the tests were made under the writer's direction, so that he can vouch for them. In a few exceptional instances, coefficients at high velocities were obtained, reaching from 0.99 to unity for 6 and 8-in. tubes. In no instance were coefficients found high at low velocities and low at high velocities. The writer's tests of tubes made by other manufacturers, in all their characteristics, agree substantially with those of his own design and make.

TABLE 3.—TESTS FOR ACCURACY OF SIMPLEX VENTURI TUBES.

Item.	Throat Velocities, in Feet per Second.								
	1.	2.	4.	6.	8.	10.	20.	25.	30.
(*a*) SIMPLEX ADOPTED COEFFICIENTS									
Coefficient, *c*.	0.934	0.949	0.964	0.969	0.972	0.972	0.976	0.977	0.977
(*b*) PERCENTAGE OF ERROR OF SIMPLEX COEFFICIENTS									
Based on average of all tests, Series 26	+1.64	+0.85	+0.73	+0.52	+0.41	+0.31	+0.51	+0.41	+0.31
Based on Series 27	+1.18	+0.32	+0.42	+0.21	+0.10	+0.10	+0.31	+0.41	+0.31
Based on Series 28	+0.86	−0.52	−0.41	−0.41	−0.31	−0.41	−0.20	−0.10	0
Based on Series 29	+0.21	−0.73	−0.41	−0.51	−0.41	−0.41	−0.31	−0.10	−0.10
Based on Series 30	+0.43	−1.15	−0.82	−0.72	−0.61	−0.61	−0.31	−0.10	0
Based on Test No. 9	.....	−1.96	−1.33	−1.02	−0.82	−0.82	−0.61	−0.51	−0.51
Based on Test No. 12	−1.79	−1.56	−1.23	−1.63	−1.82	−1.92	−1.31	−1.21	−1.21
Based on Test No. 13	±0.43	−2.17	−1.43	−1.23	−0.92	−0.92	−0.81	−1.01	−1.21

In a few cases, where the writer's Venturi tubes showed coefficients of 0.99 up to more than unity, it was concluded that the results for some reason were unduly high. Assuming that this might be caused by an incorrect Venturi head, due to vortex action at the up-stream section, straightening vanes were inserted, and immediately the results came down to normal. This is a further reason for skepticism concerning coefficients greater than about 0.98.

In Table 2 are given the results of various tests made under the writer's direction. Tests Nos. 8 and 9 were made at Cornell University by E. W. Schoder, M. Am. Soc. C. E., of Cornell, and Professor Pardoe, of the Uni-

versity of Pennsylvania. All the others were made at the Hydraulic Laboratory of the University of Pennsylvania by Professor Pardoe. Tests Nos. 20 and 21 were made with tubes manufactured by the Builders Iron Foundry. Test No. 16 was made with a Venturi tube having its invert in a single straight line, the angles of convergence and divergence being at the top, 21° and 13°, respectively, and the cross-sections being circular throughout.

The Venturi meter register designed by the writer was made in 1912 to conform with the results of tests of three tubes, 16, 8, and 4-in., with a ratio, R, of approximately 2, and half angles of 10½° up stream and 2½° down stream, in each case. No change in the design of this vital part has been made since that date; the subsequent tests have verified and confirmed the original design.

In Table 3 are given percentages of error which are conclusive evidence that the coefficients adopted were justified. In the tests of Table 3 the nominal and not the exact diameter of the Venturi tubes has usually been given, but in plotting the coefficients the exact diameter was used.

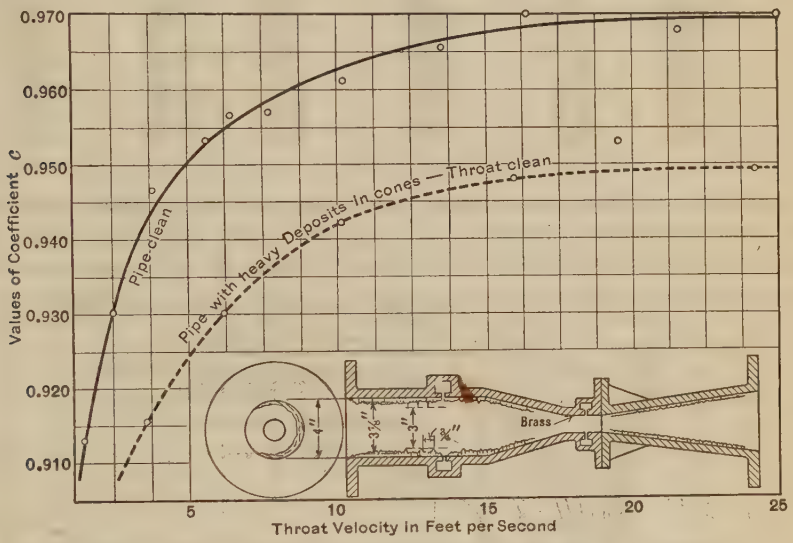

FIG. 1.—TEST OF 4 BY 1 17/32-INCH VENTURI TUBE, WITH HEAVY DEPOSIT IN CONES, THROAT CLEAN.

Test No. 22 (Table 2) was made with a clean tube. This tube had been considerably tuberculated, and a test of it was also made before cleaning. The tests gave the coefficients shown in Fig. 1 and in Table 4.

The loss of head in the dirty tube for a 25½-ft. velocity through the throat was 12.8 ft., and for the clean tube, 2.5 ft. This enormous difference did not interfere seriously with the accuracy, however, the difference being less than 2½ per cent.

The loss of head across the Venturi tube has generally been taken as the difference in piezometric heads between the up-stream portion of the tube and the point at a distance approximately three diameters below the down-stream end of the tube. This is well shown in Fig. 2.

TABLE 4.—Tests Showing Effect of Cleaning Venturi Tubes, on the Coefficient of Discharge, c.

Run.	H, in feet.	Theoretical velocity.	Flow, in cubic feet per second.	V_1, in feet per second.	c.	f, in feet, per tube plus 18 in. of 4-in. pipe.
			(*a*) Tube with Heavy Deposits in Cones			
1.....	0.2519	4.07	0.004775	3.73	0.916	0.296
2.....	0.6844	6.71	0.0798	6.24	0.930	0.850
3.....	1.770	10.77	0.01801	10.16	0.942	2.240
4.....	4.294	16.78	0.2036	15.92	0.948	5.450
5.....	6.380	20.47	0.2497	19.52	0.953	8.150
6.....	9.904	25.50	0.3097	24.18	0.949	12.800
			(*b*) Tube Clean			
1.....	0.04955	1.804	0.0211	1.648	0.913	0.0166
2.....	0.11115	2.703	0.03218	2.515	0.930	0.030
3.....	0.25111	4.065	0.04925	3.845	0.9465	0.070
4.....	0.53572	5.935	0.0724	5.660	0.953	0.148
5.....	0.6976	6.770	0.0830	6.480	0.9565	0.178
6.....	1.005	8.130	0.09965	7.780	0.957	0.302
7.....	1.733	10.67	0.1312	10.25	0.961	0.406
8.....	2.997	14.03	0.1736	13.56	0.966	0.730
9.....	4.345	16.89	0.2100	16.41	0.970	1.045
10.....	7.550	22.26	0.2760	21.55	0.968	1.760
11.....	10.065	25.73	0.3197	24.98	0.970	2.520

In addition to those tests already noted, one on a 30 by 18-in. tube was made under the writer's direction at Cornell University (February, 1926), by Professors Schoder and Pardoe. The capacity—about 58 cu. ft. per sec.—was

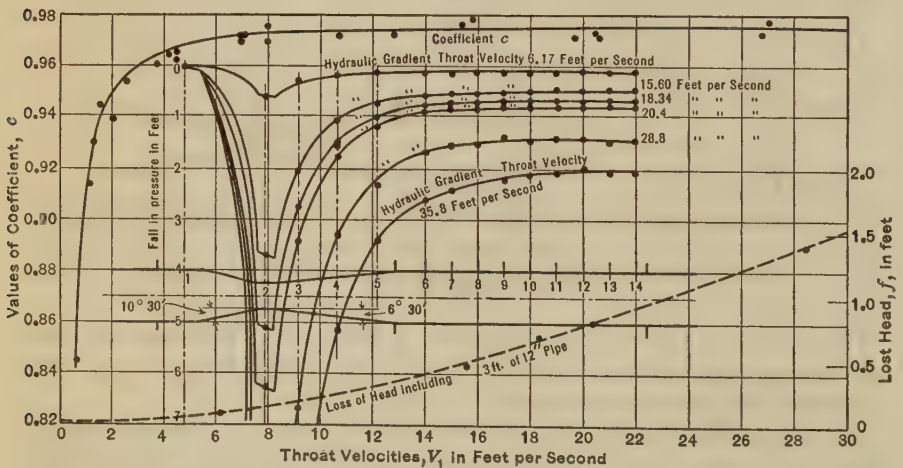
Coefficient c
Hydraulic Gradient Throat Velocity 6.17 Feet per Second
15.60 Feet per Second
18.34 " " "
20.4 " " "
28.8 " " "
Hydraulic Gradient
Throat Velocity
35.8 Feet per Second
10° 30'
6° 30'
3 ft. of 12" Pipe
Loss of Head including
Fall in pressure in Feet
Values of Coefficient, c
Lost Head, f, in feet
Throat Velocities, V1 in Feet per Second

Fig. 2.—Test of 12 by 6-Inch Special Venturi Tube, Showing Values of c, Hydraulic Grade Lines, and Losses of Head.

the extreme limit of that Laboratory. The coefficients, although somewhat high for the high throat velocities, are in close agreement with those of previous tests. As this is the largest tube tested under the writer's direction at a university laboratory, the results are reproduced herewith (Fig. 3).

The coefficients for throat velocities of more than 5 ft. appear to be somewhat high, which may be caused by slight errors in the observed Venturi head due to vortex action at the up-stream point of the Venturi tube. The arrangement of the piping, as shown in Fig. 3, would lend color to this supposition. Possibly if straightening vanes had been inserted above the Venturi tube, the coefficients might have been somewhat reduced. However, the coefficients, except at low velocities, follow closely the curve computed by Professor Pardoe.

The conditions for making the loss of head measurements were not favorable on account of the necessity of discharging from a 30-in. into a 36-in. pipe (not shown in Fig. 3), so that the recovery of head was impaired. The results, as given in Fig. 3 and Table 5, have been corrected by calculation, on the assumption that there were three diameters (90 in.) of 30-in. pipe below the down-stream end of the Venturi tube.

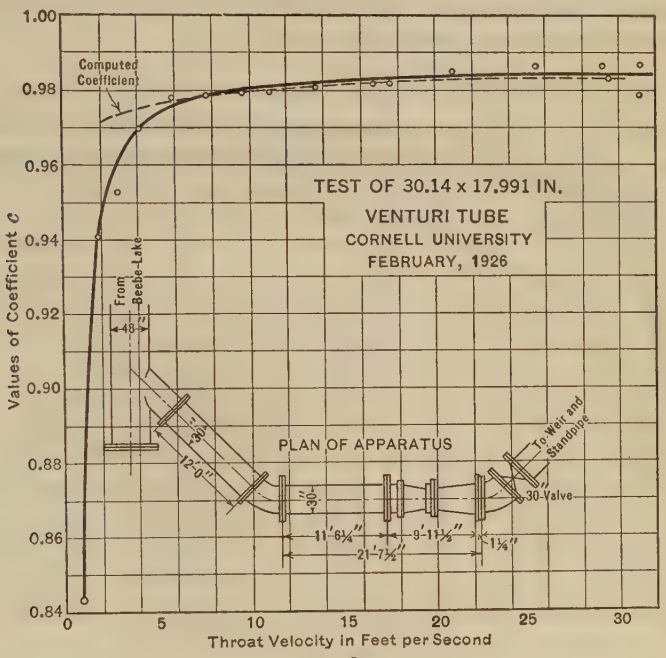

FIG. 3.

Fig. 4 shows typical curves of the coefficients for different sized Venturi tubes. It will be seen that for velocities of more than about 16 ft. through the throat, the coefficients vary from 0.96 to 0.985.

The salient conclusions are that for throat velocities, V_1, of more than 4 ft. per sec., the coefficients range between 0.96 and 0.985 for Venturi tubes more than 8 in. in diameter. For throat velocities of more than 8 ft. per sec., the coefficients are almost constant, and for throat velocities of less than 4 ft. per sec., the coefficients fall off rapidly. If due correction is made for the falling off of the coefficients at low velocities, the Venturi tube is an extremely accurate measuring device.

TABLE 5.—TEST OF 30 BY 18-INCH TUBE.

(Test No. 25-A, February, 1926; Ratio of Diameters, $R = 1.67$; Angles of Convergence and Divergence, $10\frac{1}{2}$ and $6\frac{1}{2}$ Degrees.)

Factors.	THROAT VELOCITIES, IN FEET PER SECOND.										
	1.	2.	4.	6.	8.	10.	15.	20.	25.	30.	35.
Value of coefficient, c.....	0.85	0.946	0.968	0.976	0.979	0.980	0.982	0.983	0.984	0.984	0.984
Loss of head, f.....	.....	.....	.....	.....	.....	0.34	0.78	1.10	1.48	1.83	2.26

The coefficients for very small tubes, such as 2 in. and less, have the same characteristics as for the large tubes, but are lower in value.

The loss of head across the Venturi tube, f, varies nearly as the Venturi head, H, and the square of the throat velocity, V_1.

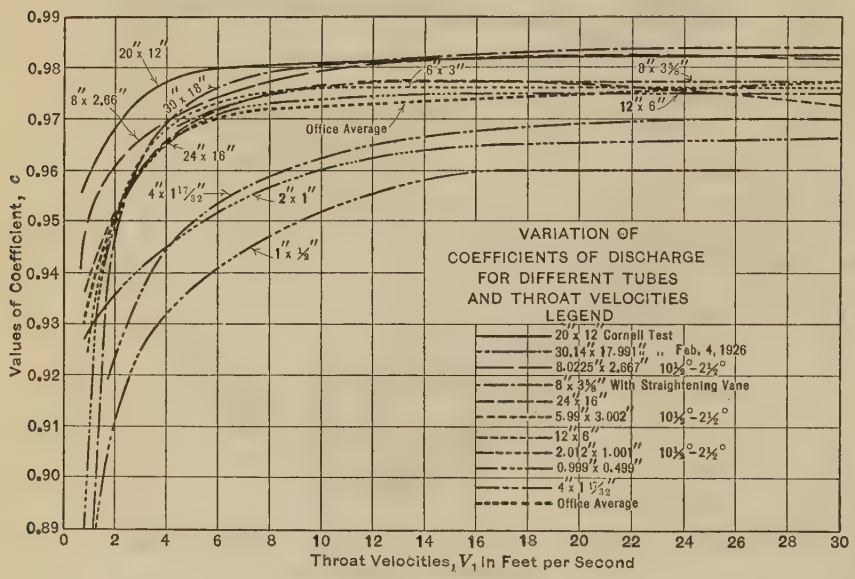

FIG. 4.

Some of the early opinions, to the effect that the Venturi tube coefficient decreases as the velocity increases, and is a maximum at the lowest velocity, have been completely disproved, the reverse being the case.

ACKNOWLEDGMENTS

Acknowledgment is due W. S. Pardoe, M. Am. Soc. C. E., under whose immediate direction the experiments at the Laboratory of the University of Pennsylvania were carried out. Acknowledgment is also due E. W. Schoder, M. Am. Soc. C. E., who, assisted by Professor Pardoe, carried out the experiments at the Hydraulic Laboratory, Cornell University, referred to in Tables 2 and 5.

DISCUSSION

J. E. Gibson,* M. Am. Soc. C. E. (by letter).—It was the writer's great privilege to be associated with the author from 1910 to 1917 during that period when the Simplex Venturi tube and registering mechanism was being developed, and to take part in the determination of the Venturi tube characteristics; and thus to become familiar with the development of the various means of measuring the Venturi differential heads. It was particularly gratifying to obtain the results of the hook-gauge method which added immensely to the accuracy of these determinations, and to learn from these experiments that the Venturi tube ratio could be materially reduced without sacrificing accuracy. Previous to this, it was the practice to make the Venturi tube ratio 9 to 1, in order to secure satisfactory accuracy, and this large ratio caused a material constant loss of head through the Venturi tube. For periodic measurements, this loss of head is not material, but for continuous records it is a serious objection. The reduction in the Venturi ratio to as low as 4 to 1 (or even lower) constituted a decided step forward in the art of fluid measurement. The nearer the area of the throat to that of the main pipe line, the more efficient becomes the pipe line for utility service, particularly fire service.

The author should be complimented on the completeness and scope of the tests as this series includes, in the writer's opinion, the most useful data on Venturi tubes yet published.

The results shown by the test of the tuberculated Venturi tube were particularly impressive. No water department is satisfied without a continuous record of the water being delivered by the plant, and while the results on new tubes are very valuable, the feature that most interests the operating man is the continuing accuracy of the meter after several years of operation when the tube and mains become tuberculated. These results show that while the loss of head through the tube may be increased as much as four or five times, the effect on the coefficient does not exceed 2½ per cent. In other words, it means that loss in accuracy of an old tube and meter, due to fouling, is less than 3%, or within the limits of observation, placing the Venturi tube as a measuring instrument superior to any other known method.

D. E. Davis,† M. Am. Soc. C. E. (by letter).—That Mr. Ledoux's paper is an exceedingly valuable and useful contribution to hydraulic literature will be especially appreciated by those who have had occasion to measure large quantities of water in connection with testing of pumps, etc., where great accuracy is at a premium. The care with which the water was weighed or measured volumetrically, and particularly the accuracy with which the differential heads on the Venturi tube were observed, render these tests most authoritative and inspire entire confidence in the results.

It would appear that the coefficients found in these experiments may be applied with assurance to any well-designed Venturi tube of similar diameter for throats of 3 in. or more. For smaller throats, variations may be found

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† (The J. N. Chester Engineers), Pittsburgh, Pa.

in individual tubes, which might develop some deviation from the coefficients found in these experiments.

The effect of deposits within the tube itself on the coefficients of discharge, is well brought out in Fig. 1. This emphasizes the necessity of a clean tube where accurate tests are to be undertaken. Most observers are familiar with similar effects in ordinary operation where proper precautions in the blowing out of mud rings and the release of air have not been taken.

The writer has been interested in comparing the results of Mr. Ledoux's latest tests with the curves of coefficients shown in the report published by the American Society of Mechanical Engineers.* Each recognizes that the throat velocities substantially determine the coefficients; and that the diameters of the pipe proper and the form of the tube exert relatively little effect. Each also recognizes increasing coefficients with increasing velocities, especially up to speeds of 10 ft. per sec. Beyond that point the curves shown in the report tend to rise slowly, whereas those of the paper approach a constant figure. Mr. Ledoux shows consistently lower coefficients. A comparison between a 12-in. and a 1-in. throat is shown in Table 6.

TABLE 6.—Comparison of Venturi Discharge Coefficients.

Authority.	Throat diameter, in inches.	Throat Velocities, in Feet per Second.			
		4	10	20	30
Author	12	0.977	0.980	0.982	0.982
"Fluid Meters"	12	0.983	0.987	0.989	0.991
Author	1	0.945	0.960	0.965	.....
"Fluid Meters"	1	0.966	0.974	0.978	0.981

The explanation of the differences may possibly be found in Mr. Ledoux's experience with vortex action, which tends to give incorrect readings for the differential head on the meter itself. It is understood that the curves in the report on "Fluid Meters" were derived from formulas, the empirical coefficients of which depend on available experiments heretofore conducted, and it is possible that inaccuracies may have crept into some of these experiments due to head readings, vortex action, etc., which may account for coefficients higher than those of Mr. Ledoux's experiments.

The writer had occasion to observe the excessive influence of vortex action where flows were being checked by pitometer tubes set at right angles near the center of the stream. The alternate rise and fall of the fluid in the two manometers pointed to a vortex action, and a volumetric check of water as measured in a basin fully corroborated the suspicions, for a variation of several per cent. was conclusively established between various runs, depending on the manner in which the vortex action was set up.

In a recent check on a Simplex Venturi meter, the dimensions of which were 36 by 18 in., the writer had occasion to make four volumetric checks in a basin where accurate hook-gauge observations could be made. The throat velocities were about 20 ft. per sec. and the average coefficient was found

* "Fluid Meters", Pt. 1, 1924 Edition.

to be 0.983, which is identical with the results in Mr. Ledoux's Test No. 25-A for a 30 by 18-in. tube (Table 5). In these checks the maximum variation was 0.003 in terms of the average coefficient.

The author's conclusion, that with the proper choice of coefficients the Venturi tube is an extremely accurate measuring device, is fully supported, and great credit for the experiments which have contributed to this accuracy is due Mr. Ledoux.

JOSEPH A. RUSSELL,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has presented a most interesting paper and one that deserves careful reading. The tests on which it was based might at one time have been considered of interest only for a textbook on hydraulics or to a designer of metering devices, but in the preparation of the tables and in his comments on the data collected, Mr. Ledoux has made it plain that the tests are of importance to engineers in very much broader fields. In addition, the tests and the brief summaries thereof correct certain mistaken ideas formed by undergraduates.

The author's conclusions as to the design of tubes, the loss of head as affected by design, and the uniformity of accuracy for wide variations in size are important to the designer, purchaser, or user of Venturi tube equipment. The designer may obviously take more liberties to obtain the desired objectives without affecting the accuracy. The purchaser or user is provided with definite data by which to compare the accuracy of different sizes and ratios of Venturi tubes.

The series of tests reported in Table 4 are impressive. Some engineers have questioned after experiencing loss of accuracy in Venturi meters by tuberculation in the throat and mouth sections of the old, all-iron, Venturi tubes, whether better results would not be assured by providing a complete bronze up-stream section and throat. It seems apparent from this series of tests that the adequacy and extra expense for bronze throat and mouth connections are substantiated. Like many other things, however, satisfaction as regards continued accuracy is diminished somewhat by observing the loss of head for corresponding quantities, as reported in the last column of Table 4 on the runs before and after cleaning.

The writer is impressed by Mr. Ledoux's remarks as to the accuracy of the tube. As a user of this type of equipment, it is comforting to know all these important characteristics and that the manufacturers and designers go to such lengths and expense, as is evidenced by this series of tests, to substantiate calculation and design. In some cases it may make little difference to the user whether the coefficient of discharge of his Venturi tube is 0.930 or 0.970, but he should be quite concerned as to whether the designer knows this and whether or not he has accommodated his equipment to this fact.

W. S. PARDOE,† M. AM. SOC. C. E. (by letter).—This paper places before engineers data that are usually considered to be private property by firms manufacturing inferential meters.

* Engr., Penna. R. R. Water Cos., Philadelphia, Pa.
† Prof. of Hydr. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

The writer has published* the following expression, with its derivation, for the coefficient of Venturi meters,

$$C = \frac{\sqrt{1 - \left(\frac{d_2}{d_1}\right)^4}}{\sqrt{1 - \left(\frac{d_2}{d_1}\right)^4 + 5.75 \left\{ f_1 \frac{l_1}{d_1} \left(\frac{d_2}{d_1}\right)^4 + f \frac{m^5}{4\, d_2} \left(\frac{1}{m^4} - \frac{1}{(m + l)^4} \right) + f_2 \frac{l_2}{d_2} \right\}^{1.25}}}$$

f_1, f, and f_2 are the coefficients of friction for clean cast-iron pipe used by the late Hamilton Smith, M. Am. Soc. C. E. The other notation is shown on Fig. 5.

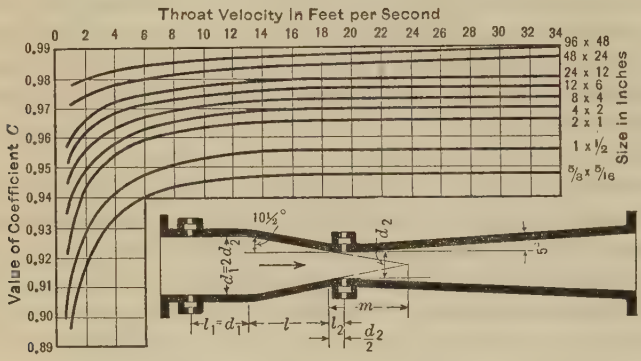


FIG. 5.

The formula was obtained semi-empirically from data of tests of Venturi meters from $\frac{5}{8}$ by $\frac{7}{16}$ in., to 16 by 8 in., and from tests of a 6-in., brass, conical nozzle with various sized tips. The close agreement of this formula and experimental results is shown by Fig. 6 for a 4 by 2-in. Venturi meter (Builders Iron Foundry) and by comparing the curve in Fig. 5 for the 12 by 6-in. meter with Figs. 2 and 3. Recent tests on larger meters have not indicated a necessity for changing the formula, but have rather confirmed it.

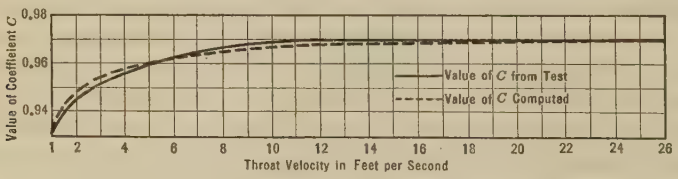

0.98
0.96
0.94
Value of Coefficient C
Value of C from Test
Value of C Computed
1 2 4 6 8 10 12 14 16 18 20 22 24 26
Throat Velocity in Feet per Second

FIG. 6.

Tests of a 4 by 2-in. Simplex Venturi meter obtained from H. P. Hammond, M. Am. Soc. C. E., at the Polytechnic Institute, Brooklyn, N. Y., give an average value of C of 0.971 from 10 to 50 ft. per sec. The tests were made volumetrically and confirm tests of Simplex Venturi meters in the Hydraulic Laboratory of the University of Pennsylvania.

* *Engineering News-Record*, September 25, 1919.

The writer found such a close agreement between the 4 by 2-in. Simplex Venturi meter and the 4 by 2-in. Builders Iron Foundry Venturi meter in the University of Pennsylvania Laboratory that he has collected coefficient curves of several Builders Iron Foundry Venturi meters tested in reliable college laboratories. These are shown in Fig. 7 and were obtained from the following sources:

(*a*) 36 by 16-in. meter at Worcester Polytechnic Institute, C. M. Allen, M. Am. Soc. C. E.

(*b*) 4 by 2-in. meter at The Harvard Engineering School, H. J. Hughes, M. Am. Soc. C. E.

(*c*) 2.94 by 1.25-in. meter at University of Toronto, Applied Science Department, Professor Robert W. Angus.

(*d*) 30 by 15-in. meter at Massachusetts Institute of Technology, George E. Russell, M. Am. Soc. C. E.

(*e*) 4 by 2-in. meter in University of Pennsylvania Civil Engineering Laboratory.

(*f*) 5 by 2½-in. meter in University of Pennsylvania Mechanical Engineering Laboratory.

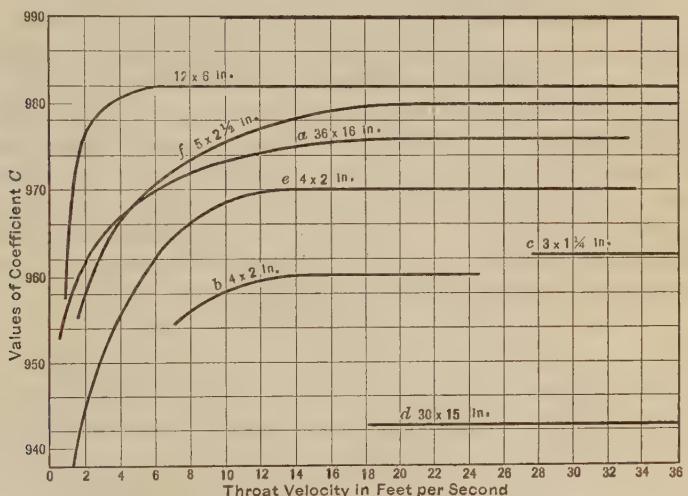

990
980
970
960
950
940
Values of Coefficient C
12 x 6 in.
f 5 x 2½ in.
a 36 x 16 in.
e 4 x 2 in.
c 3 x 1¼ in.
b 4 x 2 in.
d 30 x 15 in.
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36
Throat Velocity in Feet per Second

FIG. 7.—COEFFICIENTS OF BUILDERS IRON FOUNDRY VENTURI METERS AT VARIOUS ENGINEERING INSTITUTIONS.

Curves *a*, *b*, *c*, and *d* (Fig. 7) are about what should be expected, but Curve *d* is very low. The discharge in this case was measured by a 10-ft. suppressed weir; probably the error does not lie in this fact but rather in that the meter is placed shortly after a large side-suction centrifugal pump which discharges the water in the form of a free vortex. The result is a lowering of the coefficient, as the tangential velocity varies inversely as the radius; that is, v varies as $\frac{1}{r}$. Consequently the Venturi head would be greater than that due to the axial velocity only; hence the reduction in coefficient. That this is

possible is shown by Fig. 8 giving tests on a small brass Venturi meter, using free vortices of various angles.

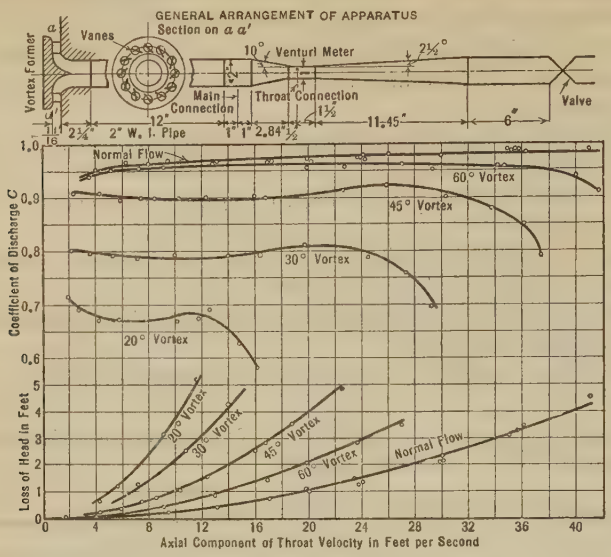


FIG. 8.—TESTS OF 2 BY 1-INCH VENTURI METER SHOWING THE EFFECT OF PASSING WATER THROUGH IT IN FORM OF A FREE VORTEX.

The writer is aware of the discussion* on this matter by Mr. A. H. Gibson, of Manchester, England, in which helical flow or a forced vortex is considered and an increase in coefficient is obtained. This the writer has also confirmed by experiment. Hence, a free vortex will decrease the coefficient and a forced vortex (v varies as r) will increase it. Incidentally, he believes the free vortex more likely to form and persist in a straight pipe than a forced vortex with high tangential velocity at the wall of the pipe.

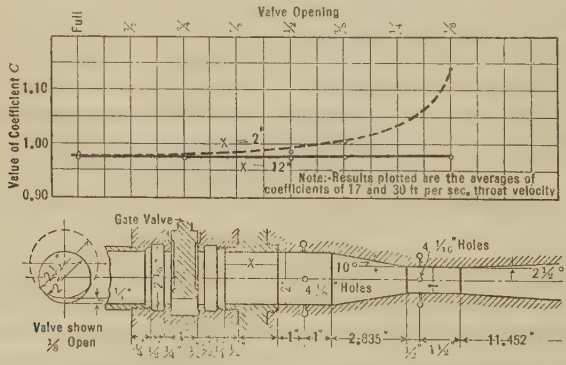

Valve Opening
Full
Value of Coefficient C
Note:-Results plotted are the averages of coefficients of 17 and 30 ft per sec. throat velocity
Gate Valve
4 1/16" Holes
Valve shown 1/8 Open

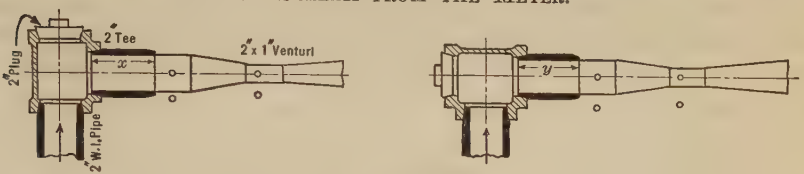
FIG. 9.—SHOWING EFFECT OF PARTLY CLOSED VALVES.

Much has been said relative to the effect of adjacent elbows and partly closed valves on the coefficient. That this has been over-emphasized is shown by Fig. 9 and Table 7.

* Abstracted in Pt. I, Report of Special Research Committee of Am. Soc. Mech Engrs., on Fluid Meters.

In all cases when the coefficient has appeared to be abnormal it has been due to a vortex, either forced or free, and the meters could be made to give correct coefficients by means of a straightening vane in their up-stream sections.

TABLE 7.—Test of 2 by 1-Inch Venturi Meter, Showing the Effect of Placing a 2-Inch Tee at Various Distances Up Stream from the Meter.



x = 2 in.		x = 4 in.		x = 6 in.		x = 10 in.		y = 2 in.		y = 4 in.	
V_t.	c.	V_t.	c.	V_t.	c.	V_t.	c.	V_t.	c.	V_t.	c.
45.6	0.9805	45.3	0.987	45.3	0.985	45.2	0.981	41.4	0.974	44.2	0.971
41.5	0.977	37.2	0.984	41.2	0.983	41.2	0.979	39.0	0.975	41.4	0.985
37.3	0.973	26.6	0.979	37.0	0.977	36.9	0.978	34.8	0.975	37.4	0.984
32.3	0.972	23.4	0 980	32.2	0.979	32.1	0.9755	29.4	0.973	32.5	0.982
29.5	0.9715	21.4	0.977	26.3	0.974	26.6	0.9765	26.5	0.9755	26.7	0.981
26.6	0.972	19.4	0.976	23.1	0.973	23.3	0.9765	23.2	0.9755	23.5	0.980
23.3	0.971	17.5	0.9755	19.3	0.974	19.3	0.977	19.5	0.973	19.5	0.980
21.4	0.970	13.7	0.975	17.2	0.976	17.4	0.975	17.0	0.9735	17.1	0.980
19.4	0.9685	10.2	0.971	13.4	0.975	13.6	0.972	13.5	0.976	13.5	0.976
17.4	0.977	8.9	0.969	8.3	0.973	8.6	0.969	9.4	0.969	8.7	0.972
13.8	0.976	....	.....	....	.....	....	.....	....	.....	....	.....
9.0	0.972	....	.....	....	.....	....	.....	....	.....	....	.....
Average ..	0.9739	....	0.9776	....	0.9769	....	0.9760	....	0.974	....	0.9791
Low*		O. K.		O. K.		O. K.		Low		Close	

* Normal coefficient, 0.977.

Mr. Ledoux arrives at the conclusion that the coefficient is 0.977 maximum, and increases with the velocity. The Builders Iron Foundry, as far as the writer knows, uses 0.99 for all sizes and velocities within the range of its recording device. The writer believes that the coefficient varies both with velocity and size of meter, although there are tests which may not confirm this.

The Venturi meter is much more accurate than the weir and slightly less accurate than the orifice. Its chief advantage over the orifice lies in the fact that the loss of head in the Venturi meter is about 25% that in the orifice meter for the same Venturi head.

Bayard F. Snow,* Assoc. M. Am. Soc. C. E. (by letter).—The correct measurement of liquids is of such importance to hydraulic engineers that all should welcome any data that contribute to the accuracy and refinement with which measurements may be made and interpreted. The question, however, of whether 0.96 or 0.985 should be used as the coefficient in a given case, although

* Res. Engr., South Essex Sewerage Dist., Salem, Mass.

apparently of considerable importance, is occasionally of relatively minor interest.

The writer has recently superintended the change of a Venturi meter tube on a sewage discharge main to prepare for the larger flows anticipated under new conditions. The old tube had been in use for twenty years. Although it was known that industrial wastes built heavy deposits in the grade sewers tributary to the pumping station, no one had thought that such formations would occur in the throat. Pumpage rates of 11 000 000 to 11 500 000 gal. per day were apparently the best that could be obtained with the present equipment, but as this meant a throat velocity of 14 ft. per sec., naturally extensive deposits were not anticipated.

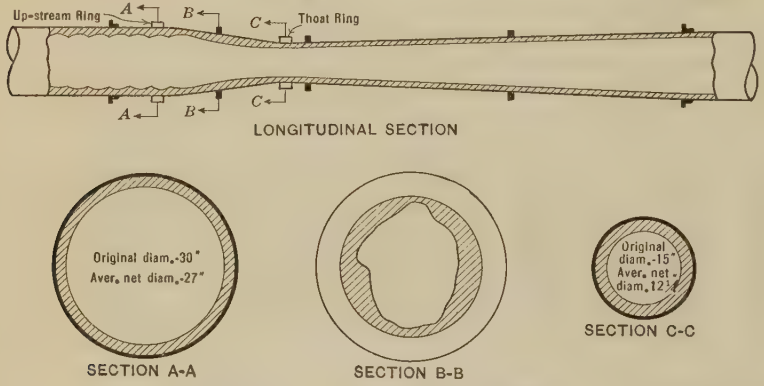


Fig. 10.—Venturi Meter Sections, Salem, Mass., Sewage Pumping Station.

When the throat section was removed a thick formation of trade wastes, consisting principally of carbonate of lime, was noticed. The condition of the meter as a whole is shown on Fig. 10. The 15-in. throat had been reduced to a smooth surfaced passage almost circular in cross-section and only 12¾ in. in diameter. The deposits up stream from the throat were thicker and more irregular. Computations based on these measurements indicated that the meter had been registering more than 50% in excess of the actual quantities pumped, or expressed differently, that a correction factor of 0.63 should be applied to recent readings. No better information being available, it has been assumed that this correction factor would decrease uniformly from 1.0 during the time the meter tube had been in service. When the deposits were removed from the part of the tube to remain in service, it was noticed that the tar coating was in perfect condition under this protection. After all changes had been completed, the register indicated that the pumpage, which under identical conditions had formerly been measured as 11 000 000 to 11 500 000 gal. per day now showed as 7 000 000 to 7 250 000 gal., thus checking the computed correction factor. This change was made in April, 1926, and recent measurements have shown deposits approximately 0.1 in. thick, of a consistency similar to that of smoothly troweled cement mortar 2 or 3 hours old.

In this problem, the question is not should a factor of 0.96 or 0.985 be used, but can the areas at the piezometer rings be maintained within reasonable limits of the original; in other words, can the Venturi tube be depended on for measurement of sewage and trade wastes?

MORROUGH P. O'BRIEN,* JUN. AM. SOC. C. E. (by letter).—The geometric similarity of several groups of the tubes listed in Table 1 gives an opportunity to check the laws of dynamic similarity. Although the tubes were probably not of as exact scale as would be used in model tests, nevertheless the results are in fair agreement with the theory. It would have helped greatly in extending the theory if other liquids had been used in addition to water and if the water temperatures had been given.

It is usually assumed that the head loss in any sort of closed pipe varies with the velocity (V), the diameter (d), and the acceleration of gravity (g). Expressing this relation in a general form,

$$f \propto V^x d^y g^z$$

in which, x, y, and z are numerical coefficients. Applying the theory of dimensions,†

$$L = L^x T^x L^y L^z T^{-2z}$$

$$\left.\begin{aligned} 1 &= x + y + z \\ 0 &= -x - 2z \end{aligned}\right\} \begin{aligned} x &= -2z \\ y &= 1 + z \end{aligned}$$

and,

$$f \propto \frac{d^{(1+z)} g^z}{V^{2z}}$$

or,

$$\frac{f}{D} \propto \left(\frac{d g}{V^2}\right)^z$$

According to the theory of dynamic similarity, if $\frac{f}{d}$ is the same for two geometrically similar tubes, $\left(\frac{d g}{V^2}\right)$ also has the same value for both tubes. Therefore, since g varies but little, the conditions for similarity are that (1) f varies with D; and (2) f and D vary with V^2. That Venturi tubes do not exactly fulfill the second requirement is shown by the values of n in Table 1, but as these values have an average value of 2 in Mr. Ledoux's general formula,

$$f = \frac{V_1^2}{470}$$

the characteristics of groups of geometrically similar tubes should show an agreement equal to the degree of approximation of this formula. In Figs. 11, 12, and 13, values of $\frac{f}{d}$ are plotted against $\frac{V}{\sqrt{d}}$ which is equivalent to reducing the head loss and velocity to values corresponding to a throat diameter of 1 in. For equal values of $\frac{V}{\sqrt{d}}$ the corresponding values of $\frac{f}{d}$ seem to be

* Research Asst., Eng. Experiment Station, Purdue Univ., Lafayette, Ind.

† "Mechanical Properties of Fluids," D. Van Nostrand, N. Y., p. 185.

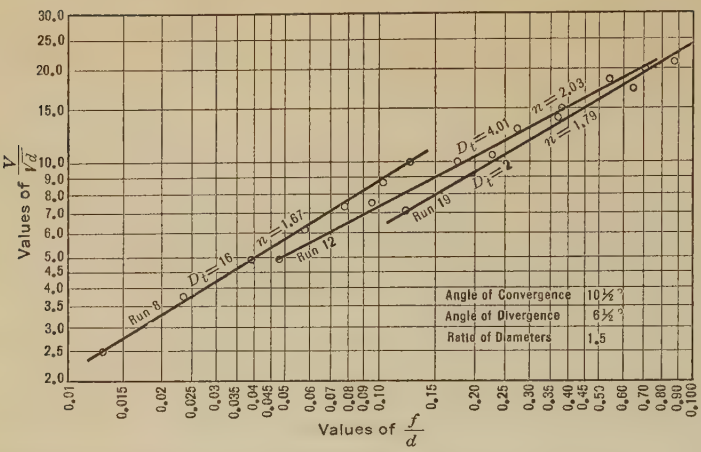


FIG. 11.

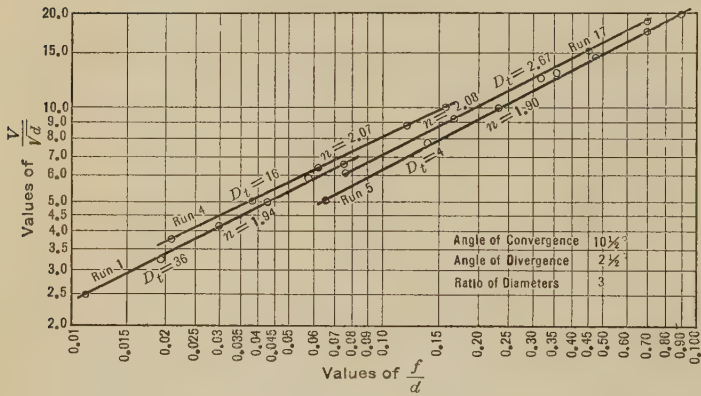


FIG. 12.

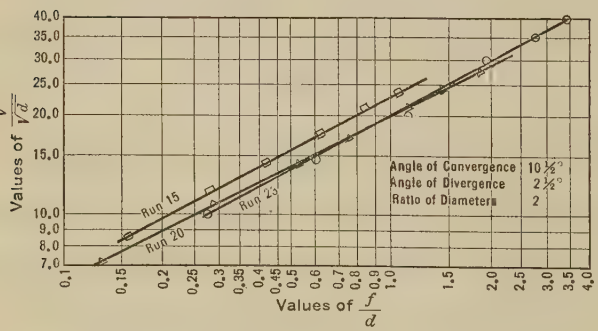


FIG. 13.

greater for the smaller sized tubes, showing a scale effect which would have to be taken into account in predicting the loss of head in a large Venturi tube. However, in Fig. 12 the curves for the 36-in. and 2-in. tubes lie between those for the 16-in. and 4-in. tubes.

In the general formula for the loss of head, the viscosity of the liquid was not considered. Including the kinematic viscosity, K, in the general expression,

$$f \propto V^x d^y g^z K^s$$

in which, $x, y, z,$ and s are unknown numerical coefficients, the relation becomes,

$$\frac{f}{d} \propto \left(\frac{D\,g}{V^2}\right)^z \left(\frac{D\,K}{V^2}\right)^s$$

The values of s and z are to be determined experimentally, but as the temperatures are not given and as the general formulas do not include d as a variable they cannot be evaluated from the values given in Table 1. The viscosity undoubtedly accounts for the scale effect, but as its influence decreases with increasing turbulence the agreement should be best at high throat velocities. This is substantiated by Figs. 11, 12, and 13, in which, with the exception of Tube No. 8, the lines seem to be converging for higher values of $\frac{V}{\sqrt{D}}$.

The quantity discharged through a Venturi meter depends on the area of the throat section, A, the operating head, h, the acceleration of gravity, g, and the kinematic viscosity, K. Writing a general equation for this relation,

$$Q \propto A^x h^y g^z K^s$$

and applying the principle of dimensional homogeneity, the equation in terms of the fundamental dimensions becomes,

$$L^3 T^{-1} \propto L^{2x} L^y L^z T^{-2z} L^{2s} T^{-s}$$

Adding powers of the same dimension,

$$3 = 2x + y + z + 2s$$
$$-1 = -2z - s$$

and the original equation becomes,

$$\frac{Q}{g\ h^{\frac{5}{2}}} \propto \left(\frac{A}{h^2}\right)^x \left(\frac{K}{g^{\frac{1}{2}}\ h^{\frac{3}{2}}}\right)^s$$

The expression, $\left(\frac{A}{h^2}\right)$, is the same for all geometrically similar tubes and $\sqrt{g\,h}$ may be replaced for V, so that,

$$\frac{Q}{A\ \sqrt{g\,h}} \propto \left(\frac{K}{V\,h}\right)^s$$

If the heads are in proportion to the other linear dimensions, h may be replaced by d, then,

$$\frac{Q}{A\ \sqrt{g\,h}} \propto \left(\frac{K}{V\,d}\right)^s$$

As $\frac{Q}{A\sqrt{g\,h}}$ is proportional to C, equal values of $\left(\frac{K}{V\,d}\right)$ should give equal values of C in all geometrically similar tubes.* If K is a constant, equal values of $(V\,d)$ should give equal values of C. Figs. 14 and 15 show such points for Runs 8, 9, 12, 19, and 25-A. Although the differences are greatly exaggerated by the scale, there is fair agreement between the curves which tend to converge for the higher values of $V\,d$. The kinematic viscosity varies 35% between 50 and 75° Fahr. and 10% between 60 and 70° Fahr., and it seems certain that some of the discrepancy would be eliminated by taking the water temperatures into account. If liquids of varying viscosities were run through these same tubes, curves of $\left(\frac{K}{V\,d}\right)$ and C could be obtained for each series of similar tubes. This is the method which Osborne Reynolds and Stanton and Pannell used in their studies of similarity in circular pipes.

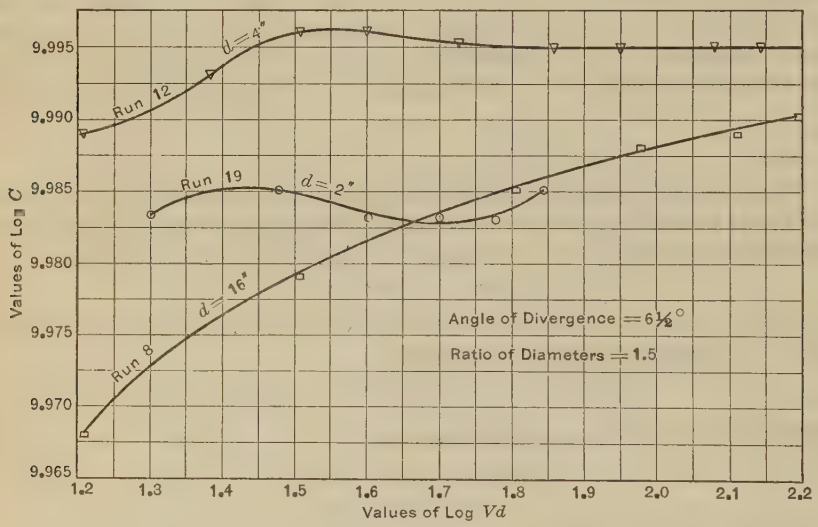


FIG. 14.

The author's Table 3 shows that the average values of the coefficient vary only 4.4% for velocities between 1 ft. and 30 ft. per sec. at the throat. This means that using an average coefficient for all velocities would give errors of only 2 per cent. Table 3 (*b*), which gives the percentage difference between the actual discharges and the discharges as computed by the average coefficients, shows that the accuracy of a Venturi tube is less than that of a carefully constructed triangular weir.

In Table 8 the values of n and a, for the different groups of similar tubes, show a rather astonishing variation. They do not seem to vary with the throat diameters, the ratio of diameters, or the angle of divergence; the only apparent relationship is that high values of a are accompanied by high values

* "Hydraulics and Its Applications," by A. H. Gibson, D. Van Nostrand Co., N. Y., p. 724; Report on Fluid Meters, Am. Soc. Mech. Engrs.

of n as would be expected. The usual assumption in regard to an equation expressing a relationship between measured quantities is that it will be a continuous function and the only continuous function which would relate n, a, and d would be periodic. Studies of pipes, orifices, and weirs do not show any reason for believing this to be the case, since the throat velocities were above the unstable region between sinuous and non-sinuous flow.

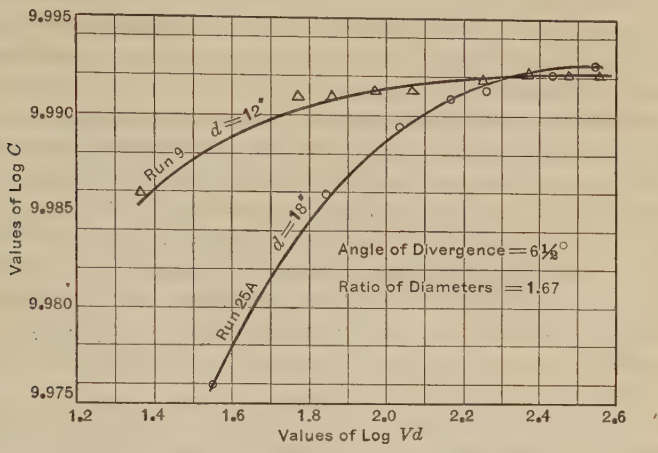


FIG. 15.

Whatever may be the cause of the variations in n, a, and c, it is evident that the Venturi tube is extremely sensitive to small changes in construction, and it seems necessary that each tube should be calibrated if an accuracy better than 1% is desired. It would be interesting to compare the performance of two commercial tubes of identical dimensions.

TABLE 8.—VALUES OF n AND a FOR DIFFERENT GROUPS OF SIMILAR TUBES.

Run No.	Ratio, R.	Throat diameter, d.	n.	a.	Angle of divergence, in degrees.
19	1.50	2.	1.79	248	6½
12	1.47	4.01	2.03	636	6½
8	1.50	16.	1.67	242	6½
23	2.01	1.	1.75	181	2½
20	1.99	2.	1.92	306	2½
15	2.	3.	1.93	388	2½
3	2.13	22.5	2.03	698	2½
2	2.11	22.5	2.02	453	2½
17	3.	2.67	2.03	603	2½
5	3.	4.	1.90	303	2½
4	3.	16.	2.07	804	2½
1	3.	36.	1.94	477	2½

Although the tests given in Table 8 do not greatly improve the status of model testing as a means of studying the performance of large-sized structures, they bring out no serious contradiction of the theory. If Tests Nos. 5 and 17 had been used to predict the performance of Tube No. 1 (Fig. 12), the error

would have been less than 10% in the discharge or loss of head; and if a previous study had shown the approximate amount of "scale effect", the error would have been much less. While a 10% error would not be permissible for a Venturi tube, it would be allowable in the case of other hydraulic structures where it is impossible to test the full-scale prototype.

ARTHUR L. COLLINS,* ASSOC. M. AM. SOC. C. E. (by letter).—For about ten years the writer has been doing test work principally in connection with the use of centrifugal pumps. In nearly every pump or turbine test the measurement of water is, of course, the most important problem. It has always been necessary to make certain assumptions regarding the proper rating of the pumps used, particularly the low land drainage pumps typical of a reclaimed country. In this particular type of installation the pump-case itself may be actually under a vacuum, the suction pipe taking water out of a pit lower than the pump, and the pump discharging also at an elevation lower than its own. It did not take long to design equipment that would measure the water accurately from the theoretical standpoint, but it took many years of experience to develop the equipment and work out a method by which readings under a vacuum could be taken almost as readily as those on a line under pressure.

A special impact or Pitot tube, covered by a patent,† has been able to produce results that have been more satisfactory than was anticipated in this class of work. It would appear that students of hydraulics have overlooked the opportunity of developing this simple type of tube. To all appearances it is merely a piece of tubing, with two small holes drilled diametrically opposite, but offset a sufficient distance so that a plug in the tube will act as a division wall. The work of calibrating the tube required several months, and was carried on in a large pump manufacturing plant where an average of two pumps per day were tested for efficiency.

In the testing laboratory of a pump company, if the method of measuring the water is incorrect, the pump manufacturer in the course of time will find it out, or at least suspect the discrepancy. Occasionally, pumps are placed in service where actual tank and volumetric measurements can be made. If the pump is not up to its rated guaranty, the volumetric test will be checked back on the method of water measurement used by the manufacturer. Few engineers are aware of the advantages of using the centrifugal pump as a means of determining quickly the calibration constants of Venturi meters, weirs, and Pitot tubes. A pump driven by a direct-connected, constant speed motor will have a definite capacity for a given head or pressure on the pump. Almost any induction motor, although not of constant speed, can be used as an accurate means of comparing measuring devices if the pump is direct-connected.

After a centrifugal pump has been set up for testing, it is extremely interesting to re-run the test, using different methods of water measurements. If there is any difference in the calibration of the measuring device, the so-called

* Cons. Engr., Berkeley, Calif.

† Patent No. 1,374,359, filed August 22, 1919, Serial No. 319,081.

characteristic curve of the pump will not repeat itself. If the true curve has been established volumetrically, then the characteristics of any other device can be quickly determined and shown graphically on the chart. The advantage in this method of comparison appears to be that the characteristic curve of the pump is definitely fixed, and the readings taken by various means of water measurement will not be affected by variations in flow. Inaccuracies due to the reading of gauges and manometers, and changes of assumed constants at various velocities will be very apparent. Under a careful and experienced engineer it is believed that Pitot tube measurements will agree within 1% of the true volumetric quantity passing at any given time.

The test data given in Table 2 would indicate that for extreme accuracy, each individual, or series, of like Venturi tubes, will agree only within certain rather broad limits. For example, in Tests Nos. 12 to 18, inclusive, covering the 6 and 8-in. range, at the 6-ft. velocity the coefficient, c, varies from 0.933 to 0.985. This means a difference of 4.8% in the constant of the meter. At the 20-ft. throat velocity, the range of the coefficient, c, is from 94.6 to 98.9, or a difference of 2.5 per cent.

These experiments might indicate that the velocity distribution of the water at the approach of the Venturi tube plays an important part in the calibration. As an approximate rule, the mean pipe coefficient, when considering the maximum velocity in the pipe, is about 85% of the maximum. This means that the velocity at the shell of the pipe is between 50 and 85 per cent. It appears that the more the uniformity of flow in the pipe is broken up, and the turbulent moving water is forced through a small orifice, the greater the consistency in the calibration of the apparatus. This is particularly true in the orifice-plate method of making measurements.

The writer has become what might be termed a "double extremist" as to the measurement of water, believing both that more accurate readings may be obtained than are generally considered possible, and also that computations may be further from the true measurement than is generally supposed, due to assuming constants gathered from miscellaneous literature.

About five years ago he was called on to develop a system of measuring syrup in a large refinery. The syrup had a specific gravity of about 1.28 and was known as the raw liquor. The pipe line was 5 in. in diameter, with velocities quite low. A monel metal orifice plate and two adjacent cast-iron sections were accurately machined and installed in the line. The calibration scale was made out in accordance with the data on the subject. Volumetric tank measurements were made and checked against the theoretically derived scale of the manometer, and the two agreed very closely. The deflections of the manometer were then recorded on a sensitive differential recorder, so that the total 24-hour run could be accurately determined.

The equipment became so important in the control of the manufacturing operations that it was decided to install the second meter on the same line, so that when the operator discovered that a mistake had been made, as shown on the chart, he would not be tempted to put the instrument out of commission or report it out of order. Identical orifice-plate installations showed a

difference of 7% in discharge under practically the same conditions. This was a case of a comparatively large orifice, and the discrepancy must be accounted for by the velocity distribution in the pipe. It was necessary to file the second plate and make the readings agree absolutely with the first plate installed, which had been consistent with the volumetric measurements. To-day, that equipment is measuring 24-hour runs and checking within about three-quarters of 1% against absolute measurements.

The writer has checked many meters by the use of the Pitot tube. Inasmuch as the Venturi tube is generally guaranteed to an accuracy of only 2½%, it has never been a difficult matter to determine whether the meter is approximately correct. Manufacturers of Venturi tubes do not differentiate between the accuracy of the Venturi tube and the accuracy of the recording and indicating equipment that goes to make up the meter unit. When called on to check a Venturi meter, it is generally for the purpose of verifying the past records. It is desirable not to disturb the Venturi unit and to take independent readings elsewhere on the line if possible, so that a corrective factor can be applied to the past record.

Many purchasers of water meters of the differential-head type do not realize that at some future date all such equipment must be "serviced". No equipment will operate indefinitely without proper care. Once a pipe line has been emptied of the water, and air allowed to be trapped in the piping leading to the registering mechanism, it is absolutely necessary that the instruments be checked in order to assure accuracy when the pipe is filled again. In checking over a meter for the purpose of correcting past records, it is advisable to consider three separate readings: First, one should observe the reading of the registering device of the manufacturer; then one should endeavor to obtain an independent manometer reading directly from the ring of the Venturi tube and determine from the deflections in the manometer and the Venturi formula, the capacity of the tube at the time. Then a Pitot tube survey will give an independent check on the quantity of water flowing at the time of the test.

This method of checking a Venturi meter is worthy of careful consideration. It is possible to check the capacity of a meter sufficiently close for commercial purposes by inserting a Pitot tube in the throat of the Venturi tube of the larger sizes. A Pitot tube of small diameter located just back of the piezometer ring in the throat, and not yet on the taper, is in a very favorable position for measurement. The velocity will be quite high, and the diameter of the pipe section is definitely known.

In California there is a steel pipe line, 30 in. in diameter, carrying both a Builders Iron Foundry meter and a Simplex meter, about 1 mile apart, with no take-off between. One meter belonged to the selling company and the second to the buying company, which would account for the incentive for the careful checking of the accuracy of the meters. In 1920 the writer made a Pitot tube survey through the throat of the Simplex meter in order to determine the accuracy of the meter. The traverse by the Pitot tube checked closely with the manometer attached to the Venturi head and the quantity calculated therefrom.

A few years later the engineer of the selling company asked the writer to check up on the Builders Iron Foundry meter. It was necessary in this case to make a Pitot tube survey on the 30-in. line as well as a manometer reading on the Venturi tube. It was found that there was a mistake in using the wrong cam in the recording mechanism of this meter. One surprising part of the pitometer survey was that it showed that the mean pipe coefficient was only 78%, rather than the generally assumed 85 per cent. However, the Pitot tube measurement, the independent manometer on the Builders Iron Foundry meter, and the independent manometer on the Simplex meter showed an agreement entirely within, or closer than the guaranty of the manufacturer.

The discrepancy that created the ill-feeling between the owners of these two meters was due to several factors. A breakage in the pipe line every few weeks would cause the line to be emptied, and when filled again air would be trapped in the Venturi meter. Also, the buyer of the water thought that he was reading in cubic feet per second when the chart showed or indicated, if he had looked closely, that it was recording in hundreds of thousands of gallons per 24 hours. The Builders Iron Foundry meter was reading in the ratio of 24 to 17 cu. ft. per sec., due to the wrong cam. After all discrepancies were smoothed out, one meter read 17.4 and the other, 17.0. This variation, 0.4 cu. ft., meant a difference of several hundred dollars in the water bill, yet the difference in the meters was only 2.35 per cent. Inasmuch as the pipe line was about a mile long, and there was a possibility that there might be a small leak in the line, the controversy was finally adjusted.

In the southern part of the State there is a Venturi tube installed on a 36-in. steel pipe line. The tube is of plate steel, and the throat apparently is a cast-iron section, with a brass-lined sleeve. The throat contraction is extremely high. The flow ranges from 1 to 30 cu. ft. per sec. The registering head was of an obsolete pattern, made by some forgotten company. No commercial meter would meet the wide range of the requirements with an accurate winter and summer flow. The summer flow would approximate a 25-ft. differential deflection.

The writer conceived the idea of double-connecting the Venturi tube. For the winter connection the Venturi tube was connected in the ordinary way. The real problem was to cut down the summer flow deflections. The deflection resulting from the high summer flow was adjusted so that it read about 25% of the previous differential. This was accomplished by tapping in on the cone section of the Venturi tube, and obtaining the differential pressure between the new leading connection and the throat connection. In determining the proper place for drilling, first the cross-sectional area was calculated from the ordinary formula, and a trial connection made. It was necessary to go several inches from the theoretically calculated location, and, in fact, the connection finally selected was very near the throat of the Venturi tube. This arbitrary piezometer connection was calibrated by means of a Pitot tube installed in the throat of the Venturi tube. In order to obtain a continuous flow record, a Foxboro differential recorder, calibrated in inches of water, was connected to the two sets of Venturi tube connections. To change from high flow to low flow, it was only necessary to open and close the valves on the connecting line.

The distribution of the velocity of the water within the pipe section at its entrance to the Venturi meter may have considerable influence on the coefficient of the meter. Venturi meters, when they show considerable variation of coefficient in tests, should be run simultaneously with a Pitot tube survey at the entrance to the meter. Obstructions in the pipe, bends, and, particularly, a centrifugal pump may interfere with the normal distribution of the pipe velocity. It may be found that this one factor will affect the coefficient, c, more than anticipated, and will account for the wide variation observed in the tests. The pipe velocity distribution also should be taken into consideration when using the color method, or the so-called salt velocity method.

It is believed that a great many constants derived from laboratory experiments and also field tests in hydraulic work, have been influenced more or less by the distribution of the velocity within the pipe, which factor has not been taken into consideration. The reason that the velocity at the shell of the pipe is not the same as at the center is due, of course, to the so-called flow resistance. The water drags at the shell surface. In a pipe where the average velocity as it approaches the measuring instrument is, say, 78%, this means an extremely low velocity near the shell of the pipe. Another pipe may show a velocity of 90 per cent. Assuming there are two Venturi tubes exactly alike, and that the approach coefficient in the one, is 78%, and in the other, 90%, it stands to reason that the coefficient, c, as determined by the manometer deflections in the pipe and the throat of the meter may not be the same. Of course, the more the original stream flow area is contracted, the more the mean pipe coefficient loses its identity and becomes a less disturbing factor in conforming to the coefficient, c, at high velocities.

Apparently, engineers are still pioneering in water measurement. Investigators should take the opportunity of determining the internal flow characteristics in any cross-section whenever possible in considering measurements by the various methods.

From Fig. 1 and noting the decreased cross-sectional area of the cone of the Venturi tube, which has an incrustation of considerable thickness, one is greatly surprised to observe in the table of tests (Table 2) the value of the coefficient, c, as determined before and after cleaning. It should be taken into consideration that the building up of an incrustation might be such that the water would be deflected at the piezometer ring so as to compensate for any error that would result from a decreased cross-sectional area. The agreement of the two sets of coefficients cannot be accounted for mathematically.

The writer does not wish to detract from the value of the information contributed, nor to question the accuracy of the results. In his own experience, circumstantial evidence indicates that tests of measuring equipment if conducted with due regard to factors not ordinarily considered, as has been pointed out, may cause the exceptional and scattered points on the curve to cling more closely to the true curve.

J. W. Ledoux.* M. Am. Soc. C. E. (by letter).—Since the paper was published complete tests of other tubes have been made, including a 2-in. tube,

* Cons. Engr., Philadelphia, Pa.

with a 1½-in. throat. For a throat velocity from 40 ft. down to 12 ft., the coefficient varied from 0.976 to 0.978; below 12 ft., the coefficient decreased to 0.912 at a throat velocity of 1 ft. A test of a 14-in. tube with an 8-in. throat was made, in which the coefficient, from 40 ft. down to 10 ft., varied from 0.981 to 0.980; below 10 ft., the coefficient decreased to 0.922 at a throat velocity of 1 ft. Both these tubes had half angles, 10½ in. up stream and 6½ in. down stream.

TABLE 9.

Venturi head, in feet of water, H.	Discharge, in cubic feet per minute, Q.	Velocity, in feet per second, through main, V.	C.	Loss of head, in feet of water.	Ratio: Loss of head to Venturi head.
4.6	7.91	96.6	174.7	1.6	0.347
4.1	7.70	94.1	180.1	1.45	0.353
3.73	7.39	90.3	181.0	1.30	0.348
3.32	7.12	87.1	185.5	1.15	0.346
2.92	6.77	82.7	187.6	1.00	0.342
2.51	6.40	78.1	191.2	0.80	0.319
2.11	6.00	73.3	196.0	0.70	0.332
1.66	5.33	63.8	196.0	0.53	0.320
1.46	5.08	62.1	198.8	0.50	0.342
1.25	4.73	57.8	200.2	0.41	0.328
0.845	3.96	48.4	203.0	....	.....
0.74	3.70	45.2	203.0	....	.....
0.422	2.83	34.6	206.0	....	.....
Average.....	....	....	.....	....	0.3377

Table 9 gives the results of a test on a ½-in. tube with a throat diameter of ¼ in., measuring air. For this tube:

$$Q = \frac{D^2 C}{3.05}\left(\frac{H}{R^4 - 1}\right)^{\frac{1}{2}}$$

$$v = C\left(\frac{H}{R^4 - 1}\right)^{\frac{1}{2}}$$

in which,

Q = quantity of air, in cubic feet per minute.
H = Venturi head, measured in feet of water.
R = ratio of diameter of main to diameter of throat.
D = diameter of main, in inches.
v = velocity of air through the main, in feet per second.
C = a constant.

It will be noted that the coefficient increases as the velocity decreases within the limits of this experiment from 96.6 to 57.8 ft. per sec. through the main.

In their discussions, Messrs. Davis, Pardoe, O'Brien, and others have added valuable data. The writer does not agree with Mr. O'Brien that the "accuracy of a Venturi tube is less than that of a carefully constructed triangular weir". Theoretically, the accuracy of a Venturi tube is more than six times as great as that of a V-notch weir measuring the same quantities

of water, which is obvious from the relation of head to flow. With the Venturi tube this is $q = m H^{\frac{1}{2}}$, and with the weir, $q = n H^{\frac{5}{2}}$, in which, q is measured in cubic feet per second, H is the head, in feet, and m and n are constants.

An error of 1% in the head produces in the triangular weir six times the error of a Venturi tube and nearly four times the error for a rectangular weir. These theoretical conclusions are fully borne out by exhaustive practical tests.

Mr. O'Brien also finds "astonishing variation" in the values of n and a in Table 8. As the accuracy of the Venturi tube is independent of the unrecoverable loss of head across the tube, these variations are not disturbing, especially as there are sufficient data to predict the performance for any particular case.

Mr. Collins describes the method of measurement with a Pitot tube to which the writer cannot object, because he has done exactly the same thing for at least sixteen years. The form of tube producing the best results is that in which the impact orifice points up stream with the low-pressure orifice at right angles, thus securing a coefficient of about 0.72 instead of about 1.00 attainable with the up and down-stream orifices at 180 degrees. This low coefficient increases the accuracy of reading by nearly 40% due to the greater suction effect of the 90°, low-pressure orifice.

For small pipes a round Pitot tube is used. For large pipes the shape is made nearly elliptical, the major axis being in the direction of flow, and providing greater resistance to impact, thus reducing the vibration of the Pitot rod.

While a Pitot-tube outfit is an excellent device for quick and economical installation for flow testing and, in many cases, for permanent installation, it can never equal the accuracy of a Venturi tube, and has the disadvantage of permitting the orifice to become clogged with débris, lint, etc., requiring frequent inspection and cleaning. Also, the main must be accurately measured and traversed before the meter register can be used.

In the hands of careful and watchful operators these devices generally give good satisfaction. Some very important Pitot tubes installed under the writer's direction have been in successful use for many years in situations where Venturi tubes could not be used within permissible expense. As to the relative accuracy and reliability of the various water-measuring devices, the writer ventures the following estimate:

	Percentage of Accuracy.
Where the water is weighed	99.9
" " " " measured in tanks	99.9
" " " " " by mechanically moving meters	99
" " " " " " Venturi tube meters	99
" " " " " " orifice meters	99
" " " " " " Pitot tube meters	97.5
" " " " " " V-notch weir meters	97
" " " " " " rectangular weir meters	96.5
" " " " " " salt velocity method	90 to 95

In regard to centrifugal pumps, there are many installations where several pumps discharge into the same main, which may have on it some form of meter, such as one of the Venturi type. One of two duplicate centrifugal pumps may be operating singly and the meter correctly measures the discharge. When two pumps operate, the man in charge is surprised to find that the discharge is not double that of one, although the speed and pressure conditions may be the same. This shows that the centrifugal pump is not a true displacement device, and cannot be used, like a positive-stroke, reciprocating pump, as an approximate check on the accuracy of the Venturi meter.

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Paper No. 1640

THE DESIGN, CONSTRUCTION, AND OPERATION OF A SMALL SEWAGE DISPOSAL PLANT*

By Franklin Hudson, Jr.,† Jun. Am. Soc. C. E.

With Discussion by Messrs. H. L. Thackwell, Webster L. Benham, L. H. Enslow, and Franklin Hudson, Jr.

Synopsis

This paper describes the design, construction, and operation of a small sewage disposal plant at Stroud, Okla., a town of approximately 2 000 inhabitants. This plant, which was designed, built, and operated for a time, by the writer, under the direction of Webster L. Benham, M. Am. Soc. C. E., has been referred to by the Sanitary Engineer of Oklahoma, as the most complete in the State.

Design

In 1920 Stroud, Okla., had a population of 1 361. This figure, however, had little bearing on the design of the sewage disposal plant as the city was growing rapidly due to the development of a sizable oil field near-by.

After considering the probable maximum future flow, and relying partly on experience in Oklahoma oil towns, it was decided to build a plant capable of disposing of the sewage from a population of 3 000. All units were arranged so that the capacity of the plant could be doubled, in the event that oil development continued.

Surveys showed that sufficient head was available between the invert of the main sewer and the flow-line of the creek to permit the use of almost any

* Published in October, 1926, *Proceedings*.

† Secy.-Treas., Benham Eng. Co., Kansas City, Mo.

gravity type of treatment plant. The type of plant selected was calculated to give the best results with a minimum of attention which in small towns of the Middle West seems to mean no attention whatsoever.

A conventional Imhoff tank, with screen chamber and sludge bed, was designed, together with a dosing tank, sprinkling filter, secondary settling tank with sludge bed, and chlorinator. During construction, a small circular brick incinerator was added, adjacent to the screen chamber, so that screenings could be shoveled directly on the grates for future burning, thus insuring a much more sanitary neighborhood in the vicinity of the screens.

The screen chamber consisted of a rectangular box having a width approximately twice the diameter of the main sewer and a length sufficient to accommodate the iron bar screen and a small operating platform. The screen proper was ¼-in. by 1½-in. bars, spaced 1½ in., center to center, on a slope of 3 horizontal to 1 vertical. The walls of this chamber were built above the top of the Imhoff tank as the small platform had been arranged to allow the entrance of sewage to the plant in case the screen should become completely clogged.

A single, reverse-flow, Imhoff tank (Fig. 1), rectangular in plan, except that the sewage troughs were rounded at the corners, was planned to care for a maximum flow of 125 000 gal. per day, based on an hourly flow equal to one-eighteenth the daily flow.

This flow, with a detention period of 2 hours and a longitudinal velocity of 0.3 ft. per min., was used to determine the dimensions of the sedimentation chamber. The sludge digestion compartment was proportioned for 1.3 cu. ft. per capita per year for a storage period of 1 year, with an additional allowance of 2½ ft. from the calculated maximum sludge level to the slots in the bottom of the sedimentation chamber.

With these data the tank was so dimensioned in plan as to allow 40% of the total area to be used for a gas vent. A slope of 1.8 horizontal to 1 vertical was used in the design of the hopper bottoms of the sludge compartment and the 8-in. sludge pipes leading to the sludge trough discharged under a head of 5 ft., measured from the normal sewage level in the tank.

The two sludge beds, with 2 in. of sand, 2 in. of fine gravel, and 8 in. of coarse gravel composing the surface, were to be of the simplest construction possible having low masonry walls to prevent the surrounding earth from washing in. It was assumed that a 1-ft. stand of sludge would be drawn on each bed ten times each year; this approximated 4 persons per sq. ft. of sludge-bed area, which was thought to be satisfactory.

Calculations showed that a dosing tank of 1 000 gal. capacity, operating under maximum conditions, would have a discharge time of about 4 min. and a filling period of 9 min., making each complete cycle of operation occupy 13 min. The height of the tank was determined from a discharge at high head, which would avoid over-spraying, and one at low head, which would be 40% in excess of the inflow. Siphon and piping losses were estimated by several formulas. For economy, and due to its position at one corner of the sprinkling filter, this tank was designed with vertical walls, which, from later

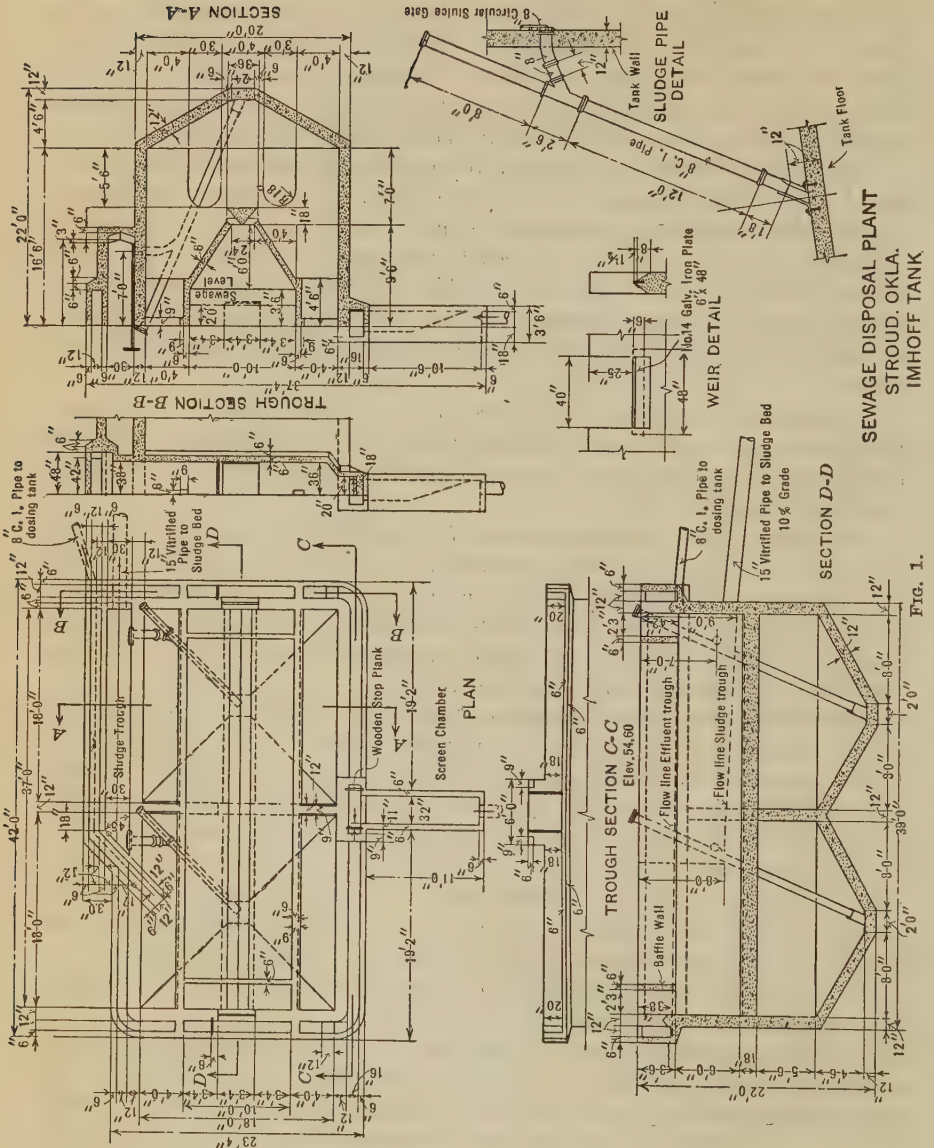
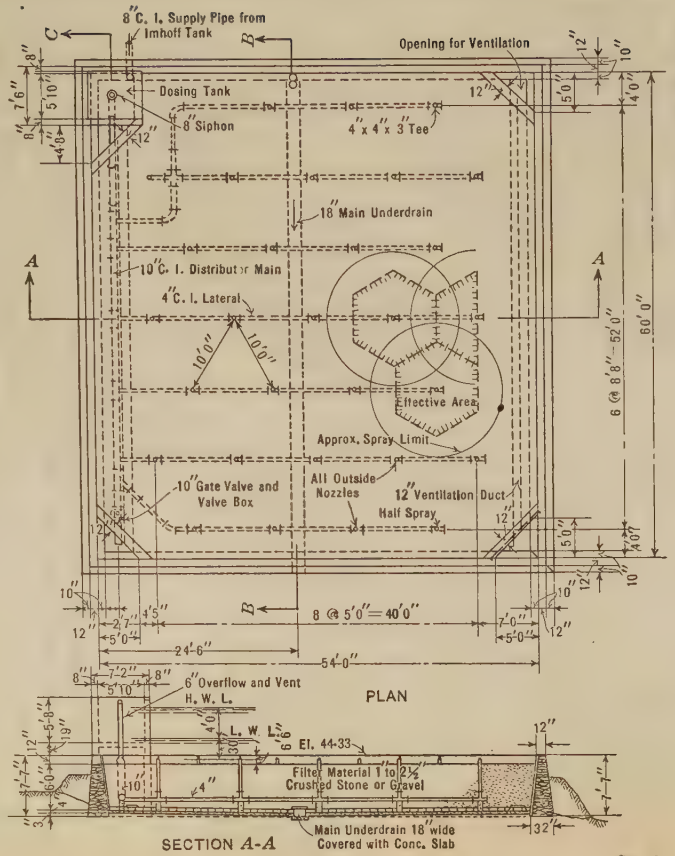
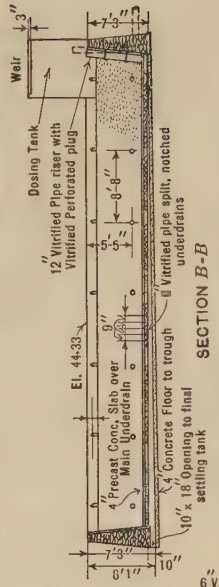
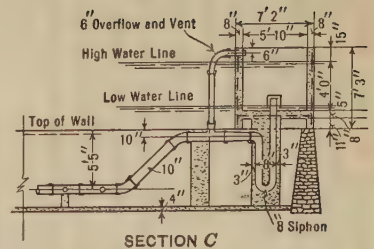
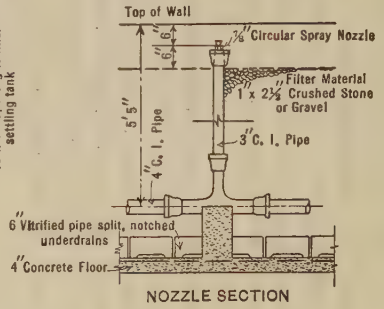

SECTION A-A
TROUGH SECTION B-B
PLAN
Screen Chamber
Wooden Stop Plank
Sludge Trough
Vitrified Pipe to Sludge Bed
8" C. I. Pipe to dosing tank
TROUGH SECTION C-C
Elev. 54.60
Flow line Effluent trough
Flow line Sludge trough
Baffle Wall
SECTION D-D
15" Vitrified Pipe to Sludge Bed 10% Grade
SLUDGE PIPE DETAIL
8 Circular Sluice Gate
8" C. I. Pipe
Tank Wall
Tank Floor
WEIR DETAIL
No.14 Galv. Iron Plate
SEWAGE DISPOSAL PLANT
STROUD, OKLA.
IMHOFF TANK

FIG. 1.




Weir
Dosing Tank
12" Vitrified Pipe riser with Vitrified Perforated plug
6" Vitrified pipe split, notched underdrains
El. 44-33
9"
4" Precast Conc. Slab over Main Underdrain
4" Concrete Floor to trough
10" x 18" Opening to final settling tank
SECTION B-B




Top of Wall
⅜" Circular Spray Nozzle
Filter Material 1" x 2½" Crushed Stone or Gravel
3" C. I. Pipe
4" C. I. Pipe
6" Vitrified pipe split, notched underdrains
4" Concrete Floor
NOZZLE SECTION

SEWAGE DISPOSAL PLANT
STROUD, OKLA.
SPRINKLING FILTER

FIG. 2.

performance, seemed quite satisfactory even if not of the most approved cross-section to give perfect distribution under varying heads.

Filter material of 1 in. to 2½-in. crushed limestone, 6 ft. deep, was chosen, with a maximum rate of filtration of 2 700 000 gal. per acre per day, as the basis for planning the sprinkling filter (Fig. 2). Spray nozzles, ⅞ in. in diameter, spaced 10 ft., gave a proper distribution of sewage. Each one of the 4-in. cast-iron pipe laterals leading from the 10-in. main served four nozzles.

Provision was made for by-passing sewage from the dosing tank direct to the secondary settling tank so that all nozzles could be cleaned and repaired. To provide proper ventilation, triangular wells were planned at each corner of the filter to which the ventilation ducts in the under-drain system were connected. These ducts were formed by omitting one section of the split, notched, under-drain tile in each line and covering the resulting opening with precast concrete slabs. The main under-drain was vented at its upper end by the use of 12-in., vitrified clay pipe with a perforated plug.

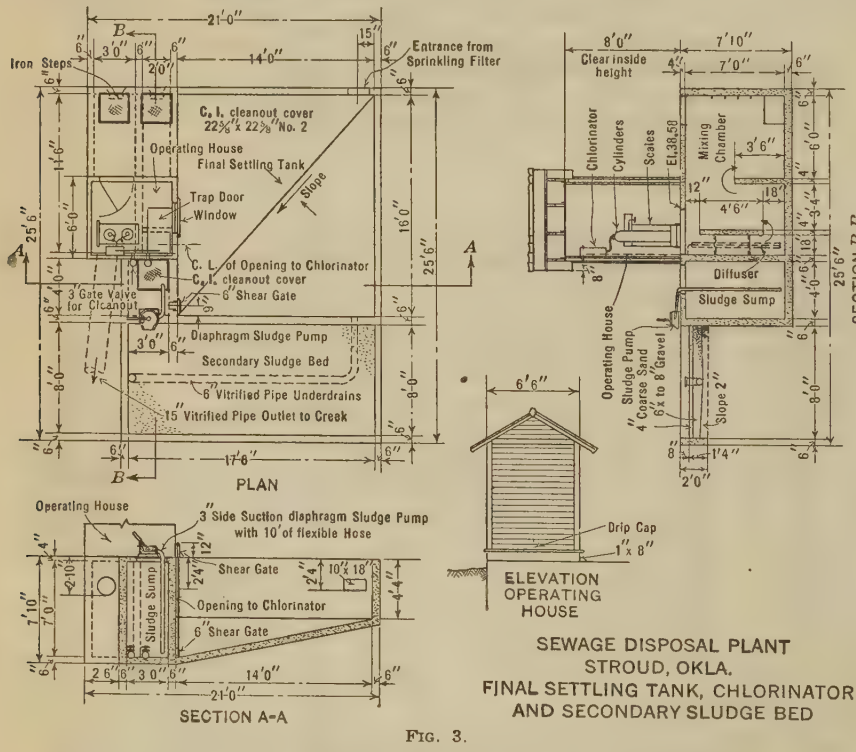


FIG. 3.

It was found that a satisfactory arrangement could be devised whereby the secondary settling tank and sludge bed, and the chlorinator (Fig. 3), could be built as one structure. The secondary settling tank was designed for ½-hour detention with 20% of the basin capacity occupied by settled sludge. No elaborate method of flow control was proposed other than to skim the liquid into the chlorinator mixing chamber at a point as remote as

possible from the entrance. Slopes of 5 horizontal to 1 vertical composed the floor of this tank.

The small secondary sludge bed was dimensioned so that with a 6-in. stand of sludge, the volume would just equal that part of the settling tank which was assumed to be occupied by sludge. Sludge was removed from the secondary settling tank by hand-pumping from a small sludge sump over which a portable diaphragm pump was placed. This small pump with flexible rubber suction hose was considered especially suitable for a plant of this type.

A standard dry-feed chlorinator housed in a substantial frame building directly over the mixing chamber gave the sewage final sterilization. The mixing chamber, with baffles to provide vertical as well as horizontal flow, was designed to hold the chlorinated liquid in detention for 20 min. before allowing it to flow into the creek.

The foregoing data are those on which the design of the plant was based and all plans were prepared.

Construction

Clearing of the site and excavating for the plant was begun early in the summer of 1924 and the plant was completed and placed in operation in the fall. The minimum of excavation was necessary as an ideal site approximately ¾ mile from the city limits had been selected.

The concrete for the hopper bottoms of the Imhoff tank and the masonry walls of the filter were placed simultaneously. There was no difficulty in constructing the Imhoff tank, the walls having been built up to the under side of all the troughs before work on the sloping sedimentation walls was started. These sloping walls were built with forms on the under side only, the concrete being poured very dry and hand-tamped into place. The walls, after having been carefully plastered, presented a smooth and true surface, in fact much better than is usually obtained by the use of forms for both sides with the resulting use of extra bracing timbers.

Following the work on the interior walls, the concrete sewage troughs and screen chamber were built, after which the sludge pipes and bar screen were put in place and the tank (Fig. 4) was ready for operation.

A suitable stone was procured locally for the walls of the filter and sludge bed. The filter walls were built to their full height, with a small section for entrance omitted, before any interior construction was started. It was decided to provide for filling the sprinkling filter with the filtered liquid to prevent small moth flies from breeding, and to do this, the walls were laid entirely in mortar and a gate-valve placed in the under-drain at the outlet. To increase the stability of the outside walls against this added load, an earthen berm was constructed within about 3 ft. of the top.

The concrete floor, siphon, and all distributor piping was placed, after which the under-drain system was laid (Fig. 5), so that hauling and dumping of filter material could be started at the earliest possible moment. This material was placed by backing the loaded wagons on a timber platform, unloading, and shoveling the material into place down the natural slope.

The secondary settling tank and chlorinator were constructed in accordance with the plans except that the height of all outside walls was increased as an added precaution against high water.

The total cost of the plant was $16 872 and, of this amount, $611 was for earth excavation. There was no classified material and no wet excavation.

Operation

During the construction of the disposal plant, the work of laying the seven miles of pipe which constituted the Stroud sewerage system had been rapidly progressing. On completion of any main or lateral sewer, permission was given to make house connections. The sewage thus acquired was cared for in a crude septic tank, formed by the construction of an earthen dam across a narrow ravine near the site of the plant, and by the time the disposal plant was ready for operation a considerable flow of sewage was available.

Normal operation began after filling the Imhoff tank, and all units (Fig. 6) functioned satisfactorily. A simple set of rules was developed to enable the plant to be operated by devoting only a part of one day each week to the task.

It was found necessary once a month to remove all nozzles from the risers and allow the dosing tank to discharge once or twice so as to remove the accumulated débris. The small incinerator was used each month and about all that was required each week was to clean the screens, check the chlorine dosage, and regulate the Imhoff tank.

Young boys were particularly troublesome before the fence around the plant was finished, and even afterward they seemed to have little trouble in getting over the "non-climbable" barbed wire. The chlorinator building was a fair target for young hunters until lurid "danger" signs were painted on each wall. Even the nozzles had an attraction, as it was discovered one morning that all of them had been broken off. After they had been replaced, however, no further damage occurred.

Excellent results were obtained during the eight months the plant was operated under the direction of the writer. Many stability tests were made, indicating a uniformly good effluent. Monthly operating costs during this period were about as follows:

Chlorine	$7.00
Plant maintenance	3.50
Sludge removal	1.50
Total	$12.00

An inspection of this plant was made after more than a year of operation by the city, and it was found to be in a condition similar to that of other plants in small towns and for which there seems to be no remedy. The screen was completely clogged, the Imhoff tank partly covered with scum, and all the nozzles on the filter bed had been removed to save cleaning them. At the secondary settling tank, the sludge was noticeable on the surface and no chlorine was being used for sterilization. On inquiry it was found that the city workmen had made only three or four trips to the plant after the writer

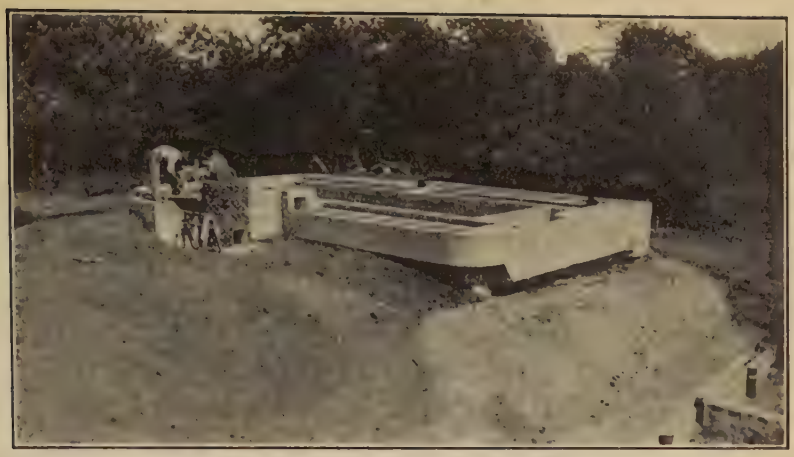


FIG. 4.—IMHOFF TANK, STROUD, OKLA.

FIG. 5.—CONSTRUCTION OF SPRINKLING FILTER, STROUD, OKLA.

FIG. 6.—DOSING TANK AND SPRINKLING FILTER, STROUD, OKLA.

had left. After a general cleaning up, replacing of nozzles and new chlorine cylinder, the plant once again functioned properly.

Conclusion

From the knowledge gained by operating this sewage disposal plant, the writer suggests the following changes that would tend to increase the practical efficiency:

Imhoff Tank.—The substitution of slots, on the flow line of the sewage troughs at the entrance to the sedimentation chamber, in place of the weirs as constructed, which would eliminate considerable trash that collects and settles back of the weirs, causing reversal of flow to be slightly unsatisfactory; and the addition of a 4-mesh screen over the entrance to the dosing tank, with a suitable emergency overflow, to prevent sizable floating matter from clogging the nozzles.

Sprinkling Filter.—Re-arrangement of spray nozzles so that no half sprays would be necessary, as these clogged badly and were hard to maintain.

Secondary Settling Tank.—The addition of vertical baffles to regulate flow so that a more uniform deposition of sludge would result.

These changes have been made in all the more recent plants designed by the writer and have been entirely satisfactory.

DISCUSSION

H. L. Thackwell,* Assoc. M. Am. Soc. C. E. (by letter).—This plant is typical of many similar layouts in Texas, with the exception that the topography of Texas is generally such that little head is available for a gravity plant. Consequently, sewage disposal plants along the bottom-lands, at the confluence of streams, are usually protected by levees; and gravity drainage during high water is impossible.

Imhoff tanks, usually subject to foaming, should be arranged with wash-watered troughs situated within the gas-vent area, so that by raising the sewage level the scum can be washed into the trough and out to the sludge drying beds. This has been accomplished in a crude way in Texas by forcing a rubber plunger below the Y-or T-connection in the sludge pipe and cutting off the riser sludge pipe below the surface of the sewage. When the sludge outlet valve is opened, the surface scum can be washed into the riser pipe and out into the sludge discharge pipe. Chester Cohen, Assistant Sanitary Engineer of the State Department of Texas, has proved that the use of chlorine in the influent of the Imhoff tank at Lufkin, designed by the writer, is a specific for foaming.

The septum walls of the Imhoff tank should be reinforced to withstand an unbalanced pressure, a hydrostatic head of at least 4 ft. The tension side of the slab would be on the side of sedimentation. There is a relationship between sludge area, transition area, and gas-vent area. These areas, if proportioned correctly, will control the shape of the outside walls. Fine screens placed in front of the effluent pipe are frequently required where baffles or primary screening are not effective in retaining chewing gum, watermelon seeds, matches, grease balls, etc., which tend to plug the trickling filter nozzles. At Electra, Tex., the operator spends a considerable part of each day taking balls of chewing gum out of the sprinkler nozzle. The chlorination of sewage flowing through the tank, if of sufficient dosage, has a salutary effect on the odors arising from the sprinkling nozzles.

In regard to the trickling filter and dosing chamber, it would seem that much better distribution of flow could have been obtained by building a central dosing chamber supported on columns and having a manifold main running in two directions. The influent to the dosing chamber could have been supported on a trestle. The losses of head then would have been more perfectly distributed and the dosing tank could have been hopper-shaped without interfering with the spray from the nozzles. The floor of the tank, if of ridge and valley construction, with a difference of about 10 ft. between ridge and valley (the valley being covered with a perforated split tile), would take care of the under-drainage more economically than tiles covering the entire floor area.

The secondary sludge settling basin, or humus tank, is coming into more general use in connection with trickling filters. The sludge from this tank, however, unless absolutely stable, should be returned to the primary digestion

* Cons. and Const. Engr., Jacksonville, Tex.

tank rather than directly to the sludge beds. It is to be regretted that the author failed to include any of the standard tests familiar to all sanitary engineers connected with such work. Without such tests, nothing can be said as to the efficiency of the various units constructed, or as to the general plant efficiency for reduction of suspended matter and bacteria. The idea of having an incinerator adjoining the screen chamber is a good one, and is used in only a few plants in Texas. The author is to be commended in that his plant seems to be of ample size, and should do very well for some years to come for a population of 3 000.

The writer's experience as to the efficiency of trickling filters when capitalized against their cost, is that they are not worth the money. Among the several outstanding features against the use of trickling filters may be mentioned:

1.—The large area required for filtration space;
2.—The high initial cost;
3.—The odors emanating from the sewage sprays;
4.—The flora and fauna having the trickling filter as their habitat; and
5.—The periodical sloughing of partly oxidized matter that has to be taken care of, either in humus tanks or by discharging directly into the stream.

If the interest on the investment in a trickling filter were used in analyzing the operating cost, in comparison with that of the bio-æration type, the balance would be in favor of bio-æration, even when power charges are as high as 2½ cents per kw-hr. Apparently, engineers are too ready to recommend this type of plant to small municipalities that do not otherwise employ a regular operator. It is far better policy to use a certain amount of machinery or mechanical equipment that requires the attendance of an operator at least a part of each day, than to build a type of plant that (the municipality may think) can take care of itself.

The writer has recently designed and constructed a plant of 65 000 gal. capacity, providing (1) a grit chamber and coarse racks, with an adjoining incinerator for screenings; (2) a primary sedimentation tank with a removable Monel metal coarse screen; (3) a secondary sedimentation tank with an adjustable Monel metal fine screen; (4) sub-irrigation for the effluent with a device for raising or lowering the underground water-table; and (5) final aeration and dilution in the stream with provision for chlorination if necessary. The sludge from the primary and secondary tanks, which is to be settled with lime as a coagulant, will be pumped into a separate sludge digestion chamber having a gas-proof cover and insulated side-walls and dome, from which the gas is drawn off and burned under a water-heater. The hot water is then circulated by means of copper tubing back through the interior of the sludge chamber, which treatment, in turn, causes more gas to be formed by raising its temperature. The cycle is thus repeated. The digester and heater are regulated by an automatic thermostat and temperature-recording devices and are equipped with gas-traps, pressure-regulator devices, an air inlet, and blow-off valves.

The digested sludge is dried on a sand bed with a standard "glass-over" construction, the dried sludge being removed in dump cars on an industrial track. The primary tank, pump and heater chamber, and digestion tank are housed in brick buildings. The secondary tank is surmounted by a concrete pergola, supporting brass pulleys which are used in hoisting the screens. The tank is bordered with ornamental tile walks and has the appearance of a formal sunken garden, since it is lower than the surrounding grass-covered berms. The whole plant is parked with shrubbery and evergreen vines and is protected with a Page non-climbable fence. Water at a pressure of 60 lb. per sq. in. is provided for the use of the operator in cleaning the tanks and irrigating the lawn.

The cost of this plant, including an entrance roadway, 900 ft. long, with bridges and culverts, a water pipe line, 1 200 ft. long, and the engineering and constructing fee, was $10 000. Operation has not yet been started (October, 1926), since to date only a few connections to the sewer system have been made. This odorless type of plant was deemed necessary since the site is close to the local college grounds, the boys' dormitory being only 900 ft. distant.

Webster L. Benham,* M. Am. Soc. C. E. (by letter).—No matter how carefully a sewage disposal plant is designed, nor how complete the general plan for treating sewage, it will be of little avail if the plant is not properly operated after it is turned over to the municipality. The successful operation of a sewage disposal plant, particularly in small cities and towns, is a bugaboo to the designing engineer. Until almost complete jurisdiction is given to State authorities requiring careful and constant attention by qualified operators, engineers cannot expect sewage disposal plants to operate successfully and to show the results for which they are designed.

The State of Oklahoma has on its statutes a law prohibiting the pollution of its streams, but the "teeth" of the law were removed by the Legislature when the original bill was presented. The result is that the State has no means of forcing a municipality to operate consistently and efficiently its sewage treatment works. It is almost appalling to see the damage done year in and year out to the sewage disposal plants, particularly in the small towns, by careless officials who seem to think that such plants are something to be built and forgotten. The writer knows of several plants in the State where contact beds and sprinkling filters have become beds of weeds and sunflowers, and where the plants have been almost dismantled by mischievous youngsters. There are even cases where the plants themselves have been abandoned and sewage is being emptied directly into ravines and streams rather than have any one "bothered" with any responsibility for the operation of the plant.

Such a condition would not and could not exist if the State Board of Health had proper jurisdiction and was protected by satisfactory laws and regulations. It is hoped that this condition will not continue long. The change for the better, however, will be due to the awakening of the people

* Pres. and Chf. Engr., Benham Eng. Co., Kansas City, Mo.

(and possibly the engineers) to the fact that investments of cities should be given proper protection and the health of the community preserved.

The writer has always made it a point to file with the city a complete schedule of operation of a sewage disposal plant. In the case of Imhoff tanks, these rules show ways and means and the tools necessary to clean the slides and slots of the sedimentation chamber so that the scum in the vent chambers can be broken up or removed. General instructions are also given relative to the removal and drying of sludge and for the resurfacing of the sludge drying beds.

The operator of every sewage treatment or disposal works should be provided with conical glass graduates for testing the quantity of suspended matter in the raw and settled sewage, so that from these observations the efficiency of any installation may be roughly determined. Tests for stability of the effluent and other indicative tests of a practical nature should be conducted daily by the operator. Such a thing is almost unknown in Oklahoma, simply due to the fact that few municipalities realize the importance of making such tests in order to determine how the treatment plant is operated. Then, again, there is no penalty enforced for not doing so, and the State Board of Health has neither the jurisdiction nor the funds to see that this work is done.

Another important feature of a sewage disposal plant is a measuring weir or other suitable measuring device for determining with a reasonable degree of accuracy the rate of flow of the sewage. Data on the sewage flow are valuable to a municipality from the standpoint of the operation not only of its disposal plant, but of its water-works system.

Engineers, however, have reason to be encouraged with the general results being obtained. As the various States place on their statutes laws regulating the operation of sewage treatment works, others will follow, and eventually the entire sewage practice with reference to operation and daily reports will be standardized.

L. H. Enslow,* Assoc. M. Am. Soc. C. E. (by letter).—With the exception of one or two details the writer will not discuss plant construction and operation except in so far as chlorination is concerned.

Care Given Small Plants.—One of the most striking statements made by Mr. Hudson, namely that "an inspection of this [new] plant was made * * * and it was found to be in a [neglected] condition similar to that of other plants in small towns and for which there seems to be no remedy," expresses truthfully, but sadly, the facts that engineers are facing.

Probably better results on the whole might be expected of small installations if the town scavenger were told to build a sewage plant and the consulting engineer later retained to operate the plant instead of the reverse practice as at present. The most perfectly designed and constructed plant must be given proper operating care. Until the consulting-designing engineer includes in the contract a clause which retains him to supervise operation of the new plant for the first year, discredit to the designing or supervising engineer will inevitably

* San. Engr., The Chlorine Inst., Inc., New York, N. Y.

result through shortcomings in the matter of care given the plant. The city authorities are too often told in the period of promotion of the new sewage project that once built there would be no more expense to speak of and very little attention would be required. The town officials, having listened intently to early given information about the proposed sewage plant, do not quickly lose the impressions made. The least intelligent man on the city payroll is too frequently assigned to the job of "running the sewer plant". Not many of the city employees want the job and, consequently, after neglect resulting in very foul conditions around the plant, including odors and flies, the job of operating the plant is not sought after and every one is perfectly willing to "let George do it".

To overcome such a state of affairs, to make the job of sewage plant operator more attractive, to obtain more intelligent operators, and to create the desired impression on the public and the city authorities after some months of plant operation, it is almost essential that the designing engineer have operating charge of the new plant for the first six months at least. Usually, a plant properly started, and an operator properly instructed and trained during subsequent visits, will prove to be a credit to every one interested, whereas plant neglect in the beginning results in sorrow for all concerned and too frequently the consulting engineer is held to blame, primarily because he can not be present to defend himself and state facts.

In designing new plants and preparing the annual budget to cover financing for amortization, operation, and maintenance, too frequently operation and maintenance have been neglected. The engineer should see to it that a sufficiently liberal fund is provided.

These suggestions may be termed idealistic or impractical, but if engineers and sanitarians will "preach" ideals concerning operation of plants, preferably under supervision of the consulting engineer, more towns will actually try the expedient. The plan should really prove feasible if the profession will work for its application. Leading engineering firms are already demanding such consideration from their clients. In the more progressive larger cities, this plan has been adopted already. It should be extended to the smaller towns where supervision, and at least initial direction, are probably more important than in the case of the larger cities.

The writer does not agree with the following statement made by Mr. Hudson: "* * * about all that was required each week was to clean the screens, check the chlorine dosage, and regulate the Imhoff tank." Experience indicates that even the smallest plants should have attention every 48 hours and preferably once daily. Screenings should be removed, the sloping walls of the flow chamber and slots "squeegeed", and the scum in the vents broken up thoroughly at least three times a week, if the plant is to be kept up to a condition which prevents the collection of scum from gaining such headway that it eventually becomes unmanageable and must be removed at considerable expense and with attendant nuisance.

Naturally enough, where chlorination is being depended upon as a part of the treatment, it becomes more important to make daily visits to the

sewage plant for the purpose of checking up the chlorinator operation and properly controlling the chlorination. This feature will be discussed subsequently in detail.

Structural Details.—The following comments and suggestions concerning structural details are open to considerable debate, but are worthy of careful thought by the designing engineer, and plant operators as well. The primary requisite of an Imhoff tank is the maximum obtainable sludge-chamber capacity below the slot with a minimum cost of construction. Secondly, the sloping bottom of the flow chamber should be at such an angle that the solid matters will not lodge thereon but will slide down and through the slot into the digestion chamber below.

With this detail in mind, the Engineering Profession has selected for almost universal usage, the slope of 1.5 vertical on 1 horizontal, producing a sloping bottom floor of the flow chamber which forms an angle of approximately 57° with the horizontal.

A triangular beam below (Fig. 7(*a*)), forms a projection on either side of the slot, preventing solids, gases, and liquid from entering the flow chamber through the slot and causing disturbances. At the same time the use of such a beam provides a slot on both sides, increases the ease with which solids may slide through into the digestion chamber, and theoretically provides also a more uniform distribution of solids in the sludge hoppers.

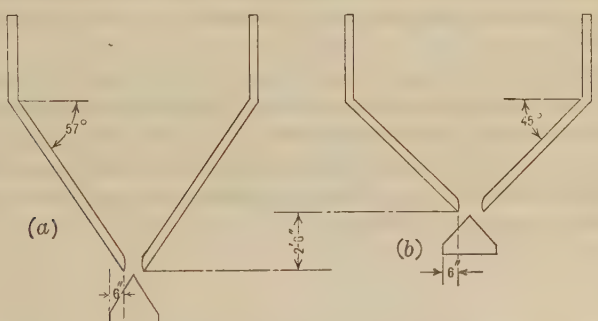


FIG. 7.—GAIN IN SLUDGE CHAMBER CAPACITY BY GIVING LESS SLOPE TO FLOOR OF FLOW CHAMBER.

To reduce in a measure the depth at which a slot would be formed when using the 57° sloping bottom, it has become somewhat common practice to project the apex of the triangular beam up into the flow chamber as in the plant at Stroud, Okla. The chief reason for such construction is theoretically a good one in that any procedure used to increase the sludge capacity of Imhoff tanks without increased depth of sludge chamber is desirable. Upon analysis, however, in many cases, it will be found that only 5 in., or at best 6 in., additional depth in sludge chamber is secured by projecting the beam into the flow chamber, and this advantage is to some extent offset by the following items.

The slots become more easily choked as a result of this beam face projecting into the flow chamber. Cleaning of the slot and the surface of the sloping

walls through effective use of the squeegee is hindered. The squeegee cannot be handled easily or effectively because the projection interferes with its passage through the slot. Another practical objection is that the size of the beam must be materially increased as to the breadth of its lower side in order to effect the proper baffling beyond the edges of the slot. This "over-lap" of the slot, which such beams are designed to produce, should preferably be as much as 6 in., if disturbances due to gas and septic liquor or solids entering the flow chamber are to be prevented. Such undesirable disturbances produce one of the most frequently encountered effects which tend to discredit Imhoff tanks.

It appears from observation of many Imhoff tanks that systematic squeegeeing of the slot and the sloping walls of the flow chamber is essential if satisfactory performance is to be expected and an effluent is to be maintained which has the least possible septic qualities and, consequently, the least oxygen demand and odor.

If, as seems to be true, squeegeeing is an essential part of routine plant operation for satisfactory results, it would appear that greater consideration might be given the idea of constructing flow-chamber bottoms with a reduced slope, with the triangular beam not projected into the flow chamber, and with a greater dependence on the practice of squeegeeing. In other words, tanks should be designed with the understanding that systematic squeegeeing is essential to their satisfactory performance, placing the burden more on operation (where it automatically falls in any event) and less, in this particular, on design.

If operation includes squeegeeing, there is the possibility of utilizing a floor slope of the flow chamber of 1 to 1 (45°) instead of the present 57 degrees. In so doing, the saving in depth becomes 2 ft. 6 in. (Fig. 7(*b*)) instead of 6 in., that is 2.5 cu. ft. of additional sludge capacity per square foot of tank area, a sizeable gain indeed, and with no interference to effective squeegeeing, as a result of the projecting beam.

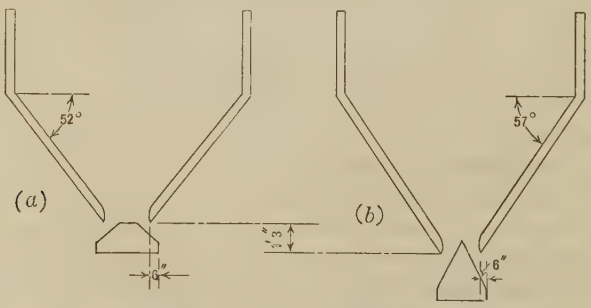


Fig. 8.—Sloping Floor at 50 Degrees and 57 Degrees, Instead of 45 Degrees.

To say the least, the difference might be split and a slope ratio of 1.25 vertical to 1 horizontal resorted to (Fig. 8), in which event the gain in sludge chamber capacity will become 1 ft. 3 in. in depth, that is, 1.25 cu. ft. of increased digestion chamber capacity for each square foot of tank area. Even this gain is worth consideration.

If an additional gain is desired, the beam might be designed as if intended to project into the flow chamber. The apex should be left out, that is, the wedge would become truncated (Fig. 8(*a*)). In this manner all the advantages of the beam projecting through the slot would be retained and the disadvantages eliminated.

At all events, the "over-lap" of slot provided through use of the beam should be as liberal as construction factors permit, preferably 6 in. or more.

Chlorination.—In designing and constructing sewage plants in which chlorination is to become a part of the process it is no longer essential to construct costly chambers to provide long contact periods between the sewage and the chlorine. In cases where some form of contact chamber appears essential, provision for a 15-min. contact, based on the average daily flow, is all that is required. Even this length of contact is not essential where chlorine dosage is regulated efficiently, but the chamber has the advantage of serving as a "balance wheel" to the operation. In other words, such a chamber takes up minor irregularities of chlorine application; in addition, the variations in the quality of effluent being chlorinated are smoothed out in passage through the contact chamber. The contact chamber, therefore, need not be efficiently baffled except at the inlet where baffling is provided to produce rapid velocity and effective mixing of chlorine and sewage.

In cases where treatment by sprinkling filters precedes chlorination, it has been shown through experiments conducted in co-operation with The Chlorine Institute, that chlorine may be satisfactorily applied to the filter effluent notwithstanding the fact that the effluent may be heavily loaded with suspended humus-like solids. Published data by Keefer* show that solids in the effluent from sprinkling filters possess a very low oxygen demand.

It is interesting to note that such solids discharged from properly functioning biological filters appear to be temporarily stabilized bodies from which the products exerting a spontaneous demand for oxygen have been removed by the oxidation processes within the filter bed. This is assumed in view of the fact that such solids possess little if any chlorine-absorbing properties (that is, chlorine demand).

It has been shown in several experiments since the summer of 1925 that, like the control of water chlorination, efficient sewage chlorination is only accomplished when sufficient chlorine has been applied (regardless of the total quantity needed) to satisfy the chlorine demand of the sewage or effluent and leave a slight quantity of residual or free chlorine in excess to attack the bacteria and spores. This being true, the measure of efficiency of chlorination of sewage effluent may be indicated by the presence or absence of residual chlorine as determined by the orthotolidin test (1 cu. cm. of standard orthotolidin solution to 15 cu. cm. of sewage sample).

The fact that not more chlorine was required to produce residual chlorine in the sprinkling-filter effluent containing the suspended solids than in the secondary tank effluent leads to the assumption and recommendation that chlorine may economically be applied at the inlet of secondary settling tanks. In following this practice the expense of construction of a special chlorination

* *Engineering News-Record*, November 25, 1926.

contact chamber can be avoided. The contact is provided and the "balance-wheel" effect secured without the additional cost that is usually considered necessary when chlorination is to become a part of the treatment. Other advantages, in that delayed fermentation of solids is effected and thus the secondary effluent is maintained in a higher state of stability and in that reversion of oxidized compounds in the effluent due to fermentation is retarded and reduced, appear probable when the secondary settling tanks are used also as chlorination or contact chambers.

In construction of secondary settling basins at small plants baffling may be utilized to distribute the solids, but a preferable method is to provide a division wall creating two long narrow chambers rather than a single tank. Such construction allows the complete and effective cleaning of one-half the unit while the other half is in service, and thus by-passing is unnecessary during cleaning operations.

In secondary tanks mechanical equipment for the continuous or frequent removal of sludge is to be preferred. In the case of the very small plants, however, a separation tank of shallow Imhoff design with floors sloping at 45° requiring weekly squeegeeing of the slopes should prove very satisfactory. The Dortmund type may likewise come in for consideration second to the Imhoff type of separation device, either of which need have but small sludge chamber capacity. The sludge may be transferred frequently to the primary tanks for digestion.

The chlorination of the incoming solids entering such tanks will not prevent satisfactory secondary digestion of these solids. This, likewise, appears to be true in the case of fresh solids entering the primary tanks where pre-chlorination of the crude sewage may be, and frequently is, more economical than chlorination of the tank effluent.

To control chlorination when applying chlorine to the influent of secondary tanks it is not necessary to maintain residual chlorine in the tank effluent. Samples of the chlorinated influent taken at or near the entrance of the tank should show residual chlorine in the supernatant liquor after allowing a 10-min. or somewhat longer period of contact and also the settling out of the heaviest suspended matter.

Chlorine Dosage Required.—No definite figure indicating the chlorine dosage required to provide residual chlorine after a 10-min. contact period can be prescribed for a particular sewage or sewage effluent. Filter effluents are more constant in their chlorine demand than tank effluents or crude sewages. In the case of tank effluents the chlorine demand varies, depending on many factors. Crude sewage is likewise subject to such fluctuations. Of all the factors the effect produced by the suspended solids present constitutes the least.

The greatest of all factors is the degree of septicity and oxygen demand of the sewage chlorinated. In this connection data published* by Keefer, of Baltimore, Md., pictures the tremendous increase in the oxygen demand of crude sewage in transit through long outfall sewers in summer and the subsequent decrease in winter.

* "Bio-Chemical Oxygen Demand of Raw and Treated Sewage," *Engineering News-Record*, November 25, 1926.

Regardless of what type or form of sewage or effluent is being chlorinated there is one universal prerequisite if chlorination efficiency is to be had, namely, regardless of the quantity necessary, residual chlorine to a very slight but positive extent (not less than 0.2 parts per million) must be found present after a 10-min. period of contact. The residual chlorine test is simple and is being used by untrained operators successfully. Fig. 9 illustrates admirably the variation in chlorine demand of a tank effluent under study by Mr. W. D. Tiedeman, of the New York State Department of Health, since October, 1925.

It is to be noted that the demand increases in the warm months and decreases markedly in the cold months only to increase again when the warm weather returns. It appears very evident from studies by Tiedeman and others that oxygen demand and chlorine demand are closely allied characteristics and that chlorine substitutes for oxygen to a great measure. The dotted line indicates the average chlorine dosage for the year which is only 9.60 parts per million. The applied chlorine during the year was distributed, however, so that satisfactory and immediate bacterial efficiency was maintained during the entire year. The dosage was high when necessary, and was reduced when the chlorine demand decreased.

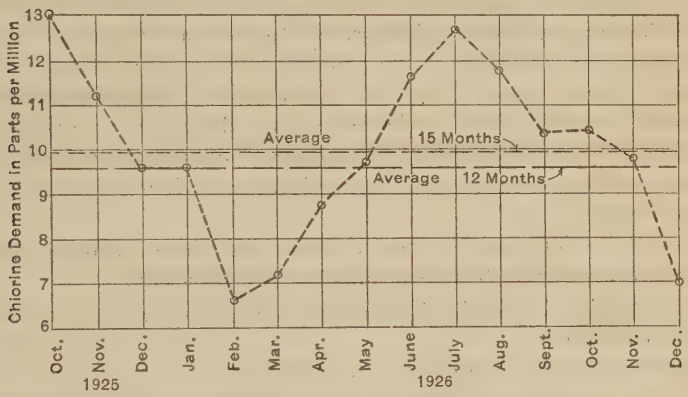


FIG. 9.—SEASONAL VARIATIONS IN CHLORINE DOSAGE REQUIRED AT HUNTINGTON, LONG ISLAND, SEWAGE PLANT.

In the past the practice of sewage chlorination has been that of applying a set dosage continuously, without regard to the variable quality of the sewage, its degree of septicity, and other factors which influence chlorine efficiency and the chlorine demand. Had 10 parts per million been continuously applied (Fig. 9), the efficiency would have been very low practically the entire year, except during the colder months, when the chlorine would have been needlessly wasted. On the other hand, had the dosage been required by the health authorities to disinfect satisfactorily under conditions of maximum chlorine demand—that is, 13 parts per million in this case—the sewage would have been over-chlorinated the greater part of the time and the cost would have been excessive.

Further work has indicated that not only does the chlorine dosage required vary during the different seasons, but likewise during each 24 hours. The night flow of sewage being weaker in organic matter requires materially less chlorine to maintain residual chlorine than the day flow. Consequently in sewage plant operation chlorine may be saved by reducing the dosage from month to month when adopting a daily control test. Moreover, added chlorine economy will result if each night the dosage is reduced to provide only the desired residual in the night flow and is increased in the early morning to care for the day flow.

In small plants this is not of importance as far as total cost reduction is concerned, but for disinfection efficiency, residual chlorine tests made daily or at least each 48 hours are essential. It is this feature, in addition to the other matters mentioned, which makes it apparent that even small sewage plants require attention from the operator more frequently than once weekly.

Pre-Chlorination of Sewage.—At sewage plants where tank treatment only is given and disinfection is required, it is frequently more economical to apply chlorine at the inlet of the settling tanks. All things being equal, regardless of the content of suspended solids, the fresher the sewage the less the chlorine demand. In plants having secondary treatment such as filters, the odor nuisance is frequently appreciable. Such odors may be reduced and finally eliminated through application of chlorine. In this case likewise the point of chlorine application should be at the primary settling tank influent or at the sewer mains. Septic action in the flow compartment is thus retarded and the odors already present in the crude sewage are destroyed or reduced. The result is a partly and, if necessary, a completely disinfected primary effluent void of odors which otherwise are liberated at the siphon chamber and from the spray nozzles.

In pre-chlorination practice, when sprinkling filters or other secondary treatment is not required, several advantages are apparent:

1.—Less chlorine is required to maintain the residual chlorine and the disinfection efficiency.

2.—The flow chamber and effluent are odorless.

3.—There is a reduction of septic activity within the flow chamber.

4.—The necessity of providing a chlorine contact chamber is eliminated as the tank serves in this capacity.

5.—From present appearances, pre-chlorination does not hinder digestion of the solids dropping through the slot and into the digestion chamber.

6.—Foaming of Imhoff tanks appears to have been controlled at certain plants through pre-chlorination.

7.—Apparently, there is an actual reduction of the oxygen demand of the effluent which can be attributed to chlorine. This is not merely a delayed (retardation of) oxygen demand, but a permanent and actual reduction varying between 25 and 50% of the demand of the unchlorinated effluent.

8.—Flocculent and finely divided solids, such as are frequently discharged in tank effluents, are penetrated and effectively disinfected when residual chlorine is maintained.

Method of Chlorine Application.—Application of chlorine through diffusors, that is, direct gaseous chlorine application, to sewage should be resorted to only when it is not practical to secure a water supply or a supply of clarified sewage for the purpose of operating the solution feed equipment.

Loss of chlorine gas escaping at the surface above the diffusors is objectionable for many reasons. Not least among these is the increased quantity of chlorine required to obtain efficient disinfection, because a good part of the gas is lost in the surrounding atmosphere. Property destruction and nuisance occur in numerous instances, also. These features add to the cost of the chlorine required and materially add to the cost of maintenance and upkeep of equipment. If the dosage of chlorine required is relatively small, as was the case at Stroud, Okla., namely, 3.5 parts per million, or less, a serious loss of chlorine above the diffusors does not occur. When larger dosages are required, such as would be true for tank effluents and crude sewage, the chlorine loss becomes an important item.

If it is essential to apply chlorine directly through diffusors, a relatively great number of them must be provided regardless of the depth of submergence below the liquid surface. In addition, the chamber in which the diffusors are placed should be sealed above to prevent any escape of the undissolved chlorine bubbles which reach the surface. In other words, provision should be made for trapping the escaped gas in a limited space above the sewage surface. Such accumulating chlorine should gradually be absorbed by the rapidly moving current of sewage in contact with it.

Feeding a solution of chlorine produces a more effective mixing of dissolved chlorine with the sewage and should likewise be expected to produce a greater efficacy with less chlorine applied.

One of the chief difficulties encountered in sewage chlorination at small plants has resulted from direct feed chlorination. The assumption has been that the equipment needed little attention. Neglect to visit the plant results in failure to detect chlorine leaks until considerable damage has been done. Likewise, the chlorinator house above the chlorination chamber becomes filled with chlorine gas which has escaped from the sewage. The result may be damaged equipment, final inoperation, and consequent lapses in continuity of disinfection.

Cost of Chlorination.—From what has been previously stated concerning variations in the chlorine demand of sewages or effluents, it becomes apparent that "each tub rests on its own bottom" and the cost of chlorination will depend on the quality of the sewage treated. One very evident fact has been established, however, that is, the better the attention and care given to maintenance of the settling tanks, the less will be the chlorine required. This is because of the more satisfactory quality of the tank effluent produced.

It can be shown that the cost of chlorination may be reduced by applying residual chlorine control. The total saving in the case of small plants is not appreciable. For larger plants it may be considerable. It has been found by Tiedeman that, volume for volume, the night sewage flow requires in some cases less than one-half the quantity required during the day. In view of the

decreased volume and coincident decreased chlorine demand a saving of chlorine results if it is reduced at night to provide only the requisite quantity of residual chlorine. In the early morning it should again be increased to insure a residual in the stronger sewage.

From the case illustrated on Fig. 9, it is apparent that without chlorine dosage control no knowledge would have been had to show that a dosage lower than 12 parts per million would prove effective for a part of the year. Consequently, to have satisfied demands that a minimum dosage of chlorine to care for all conditions during the year be maintained, would have resulted in the continuous application of not less than 12 parts per million.

With dosage control through use of the residual chlorine test applied daily at a period of maximum flow, the dosage was varied to suit conditions. The maximum applied was approximately 13 parts, and the minimum only 6.5 parts, per million. The average for the entire year was only 9.60 parts per million, that is, 3.40 parts, or 28.3 lb. per 1 000 000 gal. less than would have resulted had the constant dosage of 13 parts per million been specified as a safe minimum.

Applying this method of reduction of chlorination cost to a plant treating only 250 000 gal. per day at periods of peak flow, a saving of $(28.3 \times 0.25 \times 365)$, or 2 580 lb. of chlorine per year would result. Moreover, the sewage would be effectively disinfected under all conditions and changes in the rates of flow. Applying the same reasoning to a plant that has a peak day flow of only 5 000 000 gal. per day, the annual saving in chlorine would be 51 600 lb., which, at 5 cents per lb., amounts to a reduction in cost of chlorination of $2 580 per year.

As previously stated, the night flow of sewage reaching a plant is not only of less volume but has only one-half to two-thirds the chlorine demand of the day sewage during stages of peak flow. Assume a case of applying chlorine from 7:00 A. M. to 7:00 P. M. at a rate to maintain residual chlorine at all times, including the peak flow of 5 000 000 gal. per day. Assume that the flow at 7:00 P. M. has dropped to 3 000 000 gal. per day, with a lowered chlorine demand 66% of that of the peak day flow and that the chlorine is reduced to a rate to produce a residual at that hour and not increased again until 7:00 A. M. the following morning.

During the hours, 7:00 A. M. to 7:00 P. M., the chlorine applied would have been at the rate of (9.60×8.33), or 80 lb. per 1 000 000 gal. for 12 hours. During the night flow the rate would have been only (80×0.67), or 53.5 lb. per 1 000 000 gal.

The total 24-hour consumption, then, instead of being (5×80), or 400 lb., would have been:

7:00 A. M. to 7:00 P. M. ..... $\left(80 \times \frac{5}{2}\right)$ ..... = 200 lb.

7:00 P. M. to 7:00 A. M. ..... $\left(53.5 \times \frac{3}{2}\right)$ ..... = 80 lb.

Total for 24 hours ..... 280 lb.

The saving in chlorine resulting from the 12-hour testing and the change of dosage twice daily would have been 120 lb. per day. With chlorine costing 5 cents per lb., the annual saving effected would have been (120 × 365 × 5 cents), or $2 190.

Assume now that the plant treats a flow of 5 000 000 gal. per day (peak flow) with 12 parts per million (500 lb. per day) applied 24 hours per day continuously during the year, and compare this consumption with that of the same plant placed on residual chlorine control with only two daily tests made at 7.00 A. M. and 7.00 P. M., namely 280 lb. per day. There appears a saving of 218 lb. of chlorine per day. The cost reduction would have amounted to (220 × 365 × 5 cents), or $4 015 per year.

These illustrations have been cited merely to show that careful chlorine control may result in a reduction in the cost of chlorination, which, at the larger plants, may be sizeable. Hourly residual chlorine tests with dosage adjustments up or down barely to maintain residual chlorine in the treated sewage, will doubtlessly prove justifiable at the larger plants.

In the case of the smallest plants, chlorine dosage control maintained by making systematic tests during peak flows is justifiable and essential if efficient disinfection is to result. The saving in chlorine in such cases is unimportant because of the relatively small quantity involved. In medium sized plants, automatic chlorinating equipment should insure a reduction in the chlorine consumed. In small plants the installation of automatic equipment is rarely justified. Relatively, the chlorine saving would be small; further, the automatic equipment would require somewhat better attention and care than the manual control equipment.

In instances where the effluent from sprinkling filters is being chlorinated the fluctuations in chlorine demand during the year are naturally not expected to be as great as for tank effluents or crude sewage. However, for the purpose of obtaining disinfection efficiency with maximum chlorine economy residual chlorine tests should be applied.

At the Stroud, Okla., plant, for example, the chlorine dosage was initially 5 parts per million which, later, was reduced to 3.5 parts per million. The chlorine is applied to the effluent of the secondary settling tank. The reason for the dosage reduction was that nuisance was created as a result of excess free chlorine in the effluent. The fixed average dosage of 3.5 parts per million fails to produce the nuisance, but in the absence of residual chlorine determinations it is impossible to state that such a quantity is not wasteful of chlorine during certain periods and, likewise, that it is not deficient at others.

As is true of chlorination of potable waters, and swimming pools, it is more true of sewage, that the only effective control is that which includes systematic testing for the presence of residual chlorine as frequently as the case justifies.

FRANKLIN HUDSON, JR.,* JUN. AM. SOC. C. E. (by letter).—The writer is appreciative to those members who contributed useful and interesting information, and criticism in their discussions.

* Secy.-Treas., Benham Eng. Co., Kansas City, Mo.

The wash troughs described by Mr. Thackwell for carrying the floating scum from the gas-vent area of the Imhoff tank to the sludge drying beds should be of definite value where foaming troubles are anticipated or where constant attendance and operation are not possible. It is a question, however, whether or not such wash troughs would be used to advantage in the ordinary installation for the small municipality. In all probability, the average operator would tend to use these troughs continually rather than to stir and break up the unstable scum in order that all undigested solids might be allowed to sink to the sludge level again to "ripen" properly.

Mr. Thackwell's description of the small sewage disposal plant is very interesting, because the average sanitary engineer meets with small plant problems in ever increasing numbers.

The remarks by Mr. Benham relative to the lack of health laws, especially in Oklahoma, rigidly requiring a systematic operation of sewage treatment plants, are timely. At present, threats of lawsuits and nuisance complaints by the property owners living below the outlet of a plant seem to be the usual method of compelling the municipality to maintain its treatment plant. That such conditions will eventually be remedied is evident from the fact that an increasing number of those citizens most interested are continually bringing pressure to bear on the lawmakers at each session of the Legislature.

The writer agrees with Mr. Enslow that even the smallest plants need daily attention. However, in the case of the usual small town, with a population of less than 5 000, the always present factor of cost must be taken into consideration, and it is certainly true that thorough attention once each week, even with a possible waste of chlorine, will be less expensive than daily operation. During the period that the plant at Stroud, Okla., was operated by the writer, the following schedule of operation was carried out, and on turning the plant over to the city authorities a copy of this schedule was framed and hung on the wall of the chlorinator building and an additional copy filed with the Water and Sewer Superintendent:

"Operation Schedule
Sewage Disposal Plant, Stroud, Oklahoma,
Plant to Be Attended not Less Than Once Each Week

"Weekly Attention

"Imhoff Tank.—Thoroughly clean entrance screen with rake and empty all screenings into the incinerator.

"Skim all floating débris from the entrance to the sedimentation chamber and also stir up settlings here so that they will enter sedimentation chamber.

"Carefully scrape sloping walls of sedimentation chamber with downward movement to force all clinging particles into sludge compartment.

"Break up all scum, hard crusts and floating matter in gas vent with rake so that same will settle.

"Gauge flow of sewage from weir at outlet trough.

"Sprinkling Filter.—Remove all nozzles and clean and allow one dose of sewage to discharge before replacing. To remove nozzles it is only necessary to turn them to the left and withdraw as they are of the lock type.

"Chlorinator.—See that amount of chlorine as indicated by the instrument is sufficient to properly treat the sewage in accordance with the following:

"Depth of Sewage over Effluent Weir at Imhoff Tank.	Pounds of Chlorine per 24 Hours. (3.5 Parts per Million.)
¼ in.	0.6
½ "	1.8
¾ "	3.3
1 "	5.0
1¼ "	6.9

"Follow the instructions as outlined for chlorinator instrument in the book of instructions furnished by the Wallace and Tiernan Company.

"Once Each Month

"Sprinkling Filter.—Using pipe wrench remove all *nozzle seats* that are along east side of filter and allow sewage to discharge once, then remove remaining nozzle seats and allow sewage to again discharge. Clean, oil, and replace.

"As the sewage is discharging the first time the operator should use the rake and stir contents of dosing tank so that any settlings will pass out on to the filter.

"Every Three Months

"Sludge Bed.—Remove dried sludge from bed and replace any surface sand that has been removed.

"Imhoff Tank.—Open sludge valves, one at a time, and allow ripe sludge to flow out on to sludge drying bed. Use one bed for this.

"Secondary Settling Basin.—Using small diaphragm pump remove settled sludge from basin by pumping on to small sludge bed.

"Make an inspection trip to the plant after every rain storm."

It is admitted that the foregoing instructions are only for manual control of the plant but, nevertheless, as long as they are conscientiously followed a satisfactory sewage effluent will result.

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Paper No. 1641

STORAGE REQUIRED FOR THE REGULATION OF STREAM FLOW*

By Charles E. Sudler,† M. Am. Soc. C. E.

With Discussion by Messrs. Edward H. Sargent, Allen Hazen, Roger W. Armstrong, H. Alden Foster, Joel D. Justin, L. Standish Hall, Edward M. Craig, Jr., W. W. Brush, Richard H. Gould, A. Streiff, Sigurd Eliassen, Theodore Hatch, L. T. Guy, and Charles E. Sudler.

Synopsis

This paper presents a study followed by a set of curves for determining the dependable stream flow to be expected due to a given reservoir capacity, founded on a paper entitled "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply", by Allen Hazen, M. Am. Soc. C. E. Having found through their use in literally hundreds of instances, that Mr. Hazen's "seasonal storage curves" can be depended upon, the writer has developed extensions to them, believed to be equally reliable, for those high rates of use or draft not covered by these "seasonal" curves.

The paper also discusses those factors due to which a reservoir located on a tributary, or at a point some distance up stream from the point of use, may be less useful than one having a point of use corresponding with the dam site; and develops a series of curves for predicting the dependable flow at the point of use in all such instances. The percentage of time during which a regulating reservoir may remain only partly full, and the length of continuous periods of partial depletion of storage, are also discussed, and diagrams are presented to solve these problems.

* Presented at the meeting of February 2, 1927.
† Tarrytown, N. Y.

A number of computations are made, assuming storage capacities and rates of use as determined by the diagrams, and applying them to various streams for which flow records covering a period of years were available. These fully attest the reliability of the curves.

The New York Water Power Investigation has completed a thorough study of the water power possibilities in New York State, in the course of which it fell to the writer to study the hydrology of the various streams and the regulating effect of such reservoirs as could be found on them.

In the simplest case, involving only one practicable reservoir site, the problem was to determine its most economical capacity and the dependable flow which it would insure at the dam site. In other instances a number of good storage possibilities were found and many scattered power sites existed on the same water-shed. A further complication lay in the fact that the run-off at some power sites greatly exceeded that at the reservoirs. As the most economical capacity could only be found after comparing the regulated flows due to various capacities, through the heads affected, it was obvious that attack by the usual methods would have involved a tremendous amount of detail work. It was also realized that consistent results could not be expected where small changes in reservoir capacity or in point of regulation were involved, if the study were to depend on the ordinary use of the comparatively short-term flow records available.

The paper by Allen Hazen, M. Am. Soc. C. E., on "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply",* offered a promising tool for effecting this wholesale regulation study within the limited time available, and proved to be of the greatest value. The dependable flows indicated by Hazen's curves were checked by many direct studies of discharge records, with gratifying results. It was found that these curves gave consistent relative solutions, whereas the older method was most unsatisfactory, owing to insufficient data.

Hazen's application of the two easily derived constants, the coefficient of variation of annual flow and the storage constant, provides a simple and reliable means of ascertaining the possibilities of regulation for moderate rates of draft. The writer, in this paper, formulates a set of diagrams that are believed to be more nearly correct than those of Mr. Hazen for high rates of draft, involving large quantities of storage. Although the treatment in deriving these curves in many respects follows that of Mr. Hazen, it differs in some important details. The paper presents a discussion of the length of time and percentage of time that a reservoir will remain less than full, and a study of the usefulness of storage on a tributary.

General Considerations

The merits of the method of study here given are generally unrecognized. Few of the many reports and studies of hydro-electric development mention

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

the dependability of the proposed regulation. In most of them the matter is dismissed by stating in substance that the proposed use or draft represents the minimum flow during some period for which actual or representative hydrographs were available.

The usefulness of storage is frequently determined from a mass curve by plotting the flow to be obtained from using all the storage capacity each year. The resulting flows are then drawn as a regulated "flow-percentage of time" curve, and it is assumed that after regulation similar flow will result. To the writer such a method does not appear trustworthy: First, because the original record of flow is usually too short to portray all typical future conditions; and, second, as the flow is not known in advance it is impossible to release the storage so as to maintain steady flow without either surplus or deficiency at the end of the season. It would seem more desirable to adopt a method such as Mr. Hazen offers, by which a dependable flow is assured for some percentage of all future years, such as 90 or 95% of the time.

Run-off, deficiency, etc., are generally referred to in the following study as types which express the probability of their occurrence. Thus "90% year run-off" is that value which is exceeded during 90% of the time; "90% year seasonal storage" is that quantity which will be sufficient during 90% of the time; and "90% regulation" refers to that flow which can be dependably maintained during 90 out of every 100 years. The average annual stream flow at the point under examination is taken as unity, and drafts, etc., are expressed as fractions of this average stream flow.

Types of Regulation

Impounding reservoirs may be used in two general ways. One assumes that the storage capacity provided will be sufficient to maintain not less than some computed outflow or "draft" over a certain percentage of future years. Such regulation takes cognizance of the fact that years of low annual run-off will occur, and provides storage, if required, to make up any deficiencies during these dry years. A part of the storage may be used only occasionally; on the other hand, the reservoir may not be full throughout a period of years.

The other type, frequently used for power purposes, particularly where small storage is available, does not attempt to maintain a given minimum flow at all times, but rather to utilize all, or nearly all, the storage each year, so as to secure the maximum possible energy available with the installation.

Regulation as here contemplated provides for the release of stored water whenever the natural flow falls below an assumed rate of use, or draft, although in practical operation it is possible to operate the storage so as to secure, at times, additional flow without sacrificing the dependability of the assumed draft or minimum regulated flow.

Regulation for water supply or power purposes may serve incidentally to reduce flood flows, but unless the reservoirs provided are exceptionally large, or are operated strictly for that purpose, they do not insure complete freedom from destructive floods.

Seasonal, Annual, and Monthly Storage

Seasonal storage is a term used to measure the deficiency of flow in respect to a given rate of draft or use during some one year. It represents the quantity of water which must be released during the year to maintain the assumed draft. It does not necessarily indicate the reservoir capacity required, as may be seen in Fig. 1. In this diagram the seasonal storage required for each of the four years is shown by the ordinates *a*, *b*, *c*, and *d*, respectively, while the total storage needed for these four years is equal to *c* plus *e*.

If, during the period for which regulation is provided, the total flow of one or more years is less than the assumed draft, seasonal storage alone may be insufficient, and additional capacity, to be filled during wet years, must be provided to make good the deficiencies of the very dry years. In Fig. 1 this is shown by *e* which measures the annual deficiency of the first two years. This is called "annual" storage or deficiency.

"Monthly" storage is an expression herein used to define seasonal storage minus annual deficiency for any typical year. In Fig. 1 for the third year, the seasonal storage equals *c*, the annual deficiency is *f*, and the monthly storage is *c* minus *f*.

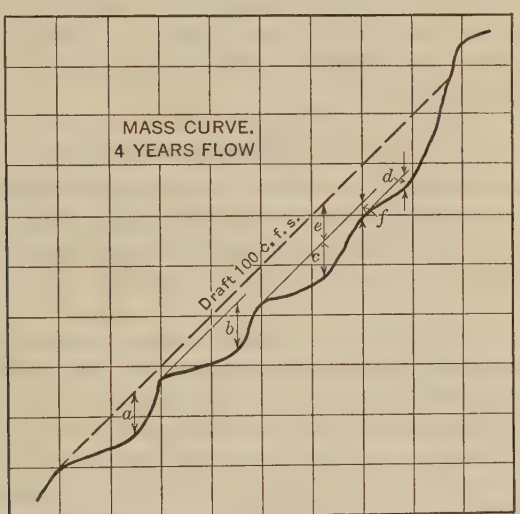

Fig. 1.

Considerations of Annual Stream Flow and Storage

Applying the theory of probabilities to the study of stream flow Mr. Hazen found it possible to classify different streams according to their habits of annual fluctuation, using an appropriate coefficient of variation (*C V*) of annual flow to express the type into which any stream would fall. Stream flow of the type represented by this *C V* might be expected to fluctuate between limits which, for any given period, increase with the increase of *C V*. Using an appropriate "run-off-percentage of time" curve, as based on the computed *C V*, the theoretical run-off of all the years of a stated period

may be ascertained, and, by chance selection of these annual values, an artificial record may be constructed.

It is thought that a long artificial record built up in this way offers a much more satisfactory means of studying annual deficiencies than such generally rather short-flow records as are available for American streams. When high rates of draft are considered, the deficiencies occasioned by a sequence of dry years are of great importance, but a flow record of even 50 years' duration does not completely cover the range and frequency of such deficiencies. For example, suppose the fluctuations of annual run-off of a stream for 10 years could be measured by ten definite values. The number of possible arrangements of these ten values is so great that a period of 36 288 000 years might elapse before any particular sequence of 10 years would be repeated!

Having assumed an "annual run-off-percentage of time" curve appropriate for the stream as expressed by its C_V, an artificial record of, say, 1 000 years' length may be built up by dividing the curve into 1 000 parts and selecting the values read at the middle of each part. A sequence is obtained by drawing the numbers representing the annual run-off values by some chance method. In the present instance fifty annual run-off values were selected and put on cards, which were shuffled and drawn one by one until the whole fifty were used. Twenty such drawings were made, thus providing a 1 000-year artificial record.

Computation of Annual Storage

In order to ascertain the probable annual storage requirements the first method tried assumed a reservoir of infinite capacity, full at the start; then, with various drafts, examination was made as to the depletion at the end of each year. These depletions were then plotted in order of magnitude as in Fig. 2.

Observing these curves one is struck by the fact that although 46% of the years have run-off greater than the average, the storage curve for a draft equal to the average stream flow (1.0) begins at 3½% of the time, and requires storage equivalent to 1.9 times the average annual run-off for 95% of the time. It is also noted that the storage required for any percentage of time is greater when the computation is based on a long record than on a short one. This is shown in Fig. 3 which embodies the same storage curves, with the addition of lines intercepting the quantities required for 5, 10, 20, 50, and 100-year periods. These values were obtained by computing the maximum storage required for each draft, for each period of 5, 10, 20 years, etc., and taking the average of these maximum values as the most probable requirement.

The inconsistencies thus seen led to examination of the artificial record with the assumption of definite storage values for the various drafts, thus approximating actual regulation. For each draft a number of reservoir capacities were assumed, and with each capacity the years of deficiency following the use of the storage, were noted.

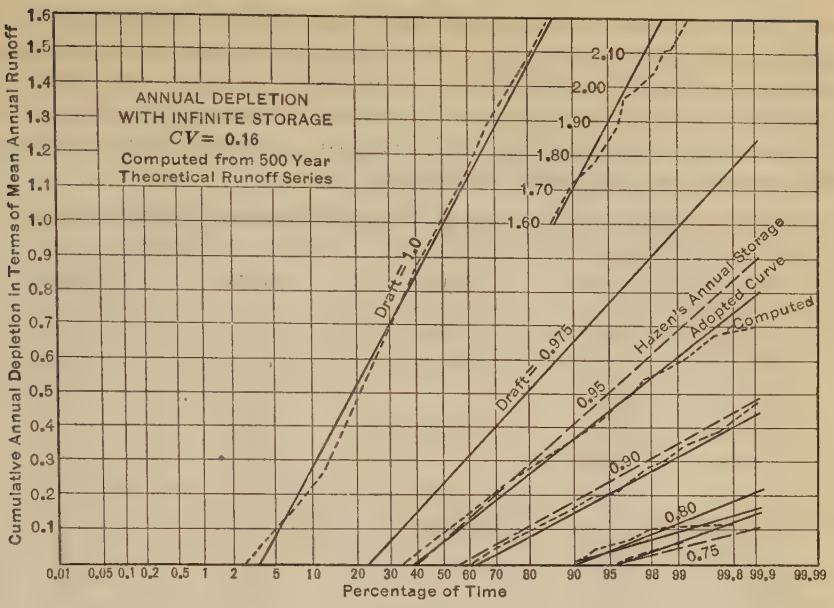

FIG. 2.

The quantities of storage ascertained by this latter method are much smaller for high rates of draft, than those indicated by the former procedure. This is entirely reasonable as may be seen in Table 1 which considers 20 years of flow.

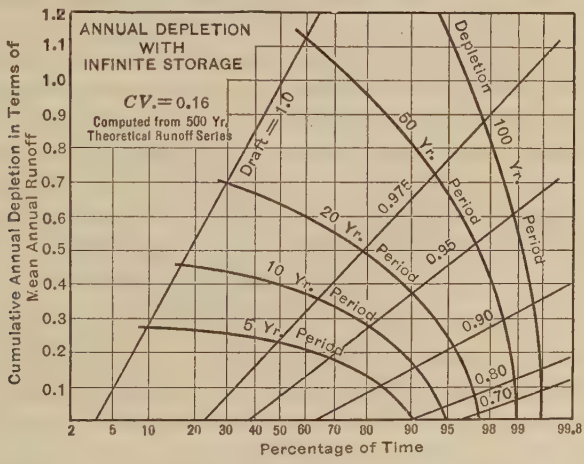

FIG. 3.

By the first method, assuming a reservoir of infinite capacity, the storage for 19 out of 20 years would be 0.69, which is the nineteenth largest value of depletion. If, however, a reservoir with a capacity of 0.44 be assumed, it is seen that the draft of 1.0 can be maintained during the first 5

years, with a deficiency of 0.25 in the sixth year. In the seventh year 0.16 of water is in the reservoir, which is sufficient to carry the draft over the eighth, ninth, and tenth dry years. In the eleventh year the reservoir is refilled to the extent of 0.29 and by the end of the fifteenth year, it is completely refilled, and is never thereafter entirely emptied. Actually, then, storage of 0.44 is sufficient for 19 out of this 20 years.

TABLE 1.—Reservoir Depletion. Draft = Average Run-Off (1.0).

Year.	Run-off.	Surplus.	Deficiency.	Depletion.
1	1.20	0.20	....	0
2	0.74	....	0.26	0.26
3	0.82	....	0.18	0.44
4	1.13	0.13	....	0.31
5	0.93	....	0.07	0.38
6	0.69	....	0.31	0.69
7	1.16	0.16	....	0.53
8	0.91	....	0.09	0.62
9	0.99	....	0.01	0.63
10	0.94	....	0.06	0.69
11	1.29	0.29	....	0.40
12	0.85	....	0.15	0.55
13	1.01	0.01	....	0.54
14	1.09	0.09	....	0.45
15	1.22	0.22	....	0.23
16	0.83	....	0.17	0.40
17	0.92	....	0.08	0.48
18	0.89	....	0.11	0.59
19	0.98	....	0.02	0.61
20	1.06	0.06	....	0.55

Graphical Use of Artificial Record—The mass curve offers a convenient means of studying storage as the cumulative flow at every point of time is shown graphically. For this study instead of plotting the accumulated flow at the end of every year, the accumulated surplus or deficiency relative to the average run-off, was used. The result is the same as if the values at every point on the usual mass curve were reduced by the product of the average flow into the number of years to each point. In this form a narrow strip of cross-section paper, as in Fig. 4, is sufficient for the diagram.

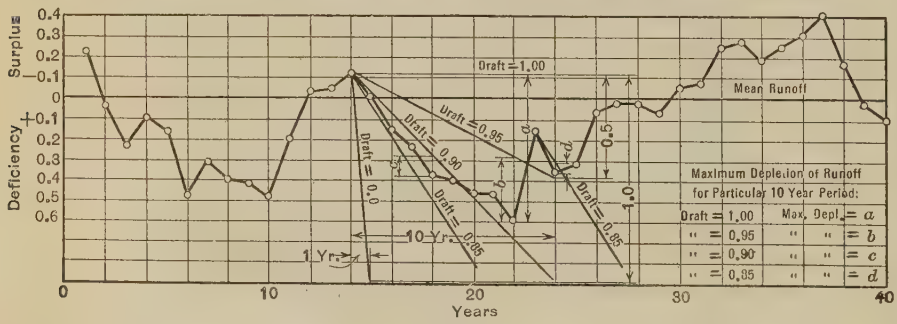

Fig. 4.—Portion of Modified Mass Curve of Annual Run-Off.

The study was made using a $C\ V$ of 0.16 for the artificial stream, although it would have been somewhat simpler to use run-off variations expressed directly in terms of $C\ V$. Smooth curves were drawn through the plotted points based on certain principles as follows:

(*a*) Annual deficiency of flow begins for any draft at that percentage of time at which the annual run-off just equals the draft. If, for example, the annual run-off exceeds 0.9, its average value during, say, 60% of the time, no annual storage will be required during this 60% of time. Thus, the beginning of each "annual storage-percentage of time" curve is located.

(*b*) When a draft is selected equal to the run-off of the dryest year of the period considered, no annual storage is required. For a small increase in draft a deficiency will occur in only one year of the period considered, and as long as this condition holds for a given increase in draft there must be an equal increase in storage. If the storage is plotted against the draft, both to the same scale, the curve must have a minimum slope of 45 degrees. The actual storage curve must then begin tangent to this 45° slope and diverge upwardly from it.

Relation of *C V* to Annual Storage

The annual storage computations were made for a theoretical stream the *C V* of which was 0.16. Hazen has shown that if storage is expressed in terms of *C V* and drafts are taken as 1—(*C V* × a constant), then for any given storage and draft, the draft will always be maintained through the same percentage of years and the deficiencies will be directly proportional to *C V*. Hence, the results obtained with the 1 000-year record can be reduced to terms of *C V*, as shown in Fig. 5.

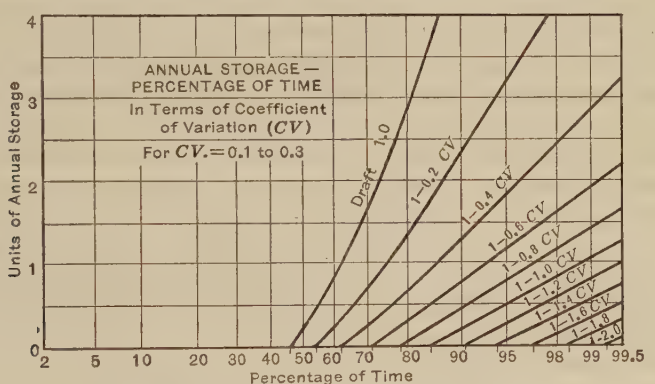

Fig. 5.

Objection has been made to this method that by increasing the value of *C V* sufficiently a depletion-draft curve will result, indicating depletion even in case of zero draft. It is seen in the theory of skew-frequency curves, however, that the *C V* must be less than one-half the coefficient of skew, as otherwise the run-off curve would require negative run-off during the dryest years. Since in this study a constant coefficient of skew is assumed, the limitation must be applied to the *C V*.

For New York State streams an average coefficient of skew of 0.60 has been selected, thus automatically limiting the *C V* of the streams consid-

ered to 0.30 or less. For any larger $C\,V$ a different curve having greater skew must be adopted.

Computation of Seasonal Storage

Seasonal storage is a measure of the deficiency in respect to a given rate of flow during single years. Hazen, in his study, which embraced the flow of streams in various sections of the United States, found, at least as far as was disclosed by the considerable amount of data at hand, that his "seasonal storage-percentage of time" and "seasonal storage-draft" curves could be applied to all streams, even to those whose characteristics of flow as shown by annual "storage-percentage of time" curves differed widely. For example, assuming a draft equal to 0.5 of the average flow, the Gunpowder River required an average storage equal to 5 days' flow, and 95% of time storage equal to 72 days' flow. The South Platte River required an average storage of 60 days' flow and 95% storage equal to 127 days' flow. In both cases the difference between the average and 95% storage is 67 days' flow at the assumed draft. The 95%-year annual run-off of the South Platte River is much less than that of the Gunpowder River, but this does not affect the relative differences between the average and the 95% requirements as expressed in day's flow at drafts that are equal proportions of the average run-off in both instances.

Expressing seasonal storage in terms of day's flow at the assumed rate of use or draft, Hazen found that a definite relation between various drafts holds for all streams; also that a definite number of days' draft would represent the difference between the seasonal storage for the average year and that for any other year. Based on these relations he computed the number of days seasonal storage required for the mean year, and the 95% year, for various rates of draft, and connected these by skew probability curves that appeared to represent the data. The actual values given these curves are greater than those of any of the streams examined, but the differences between any two rates of draft, or between the storage for one percentage of time and another percentage of time, are directly applicable to any stream. Thus, by applying a suitable constant number of days' draft, actual or definite values can be taken from the curves.

"Seasonal storage-percentage of time" curves based on Hazen's mean and 95%-year values,* connected by suitable skew probability curves are shown on Fig. 6. To apply these to a particular stream for which a few years of flow record are available, the actual average seasonal storage for one or more rates of draft is computed from the flow record in terms of the mean annual run-off. These values are compared with those indicated at the mean, or 54% year, on Fig. 6, for similar drafts, and the difference, or in case several drafts are examined, the differences, reduced to terms of day's draft and averaged, become the storage constant for that stream.

Fig. 7 based on Fig. 6, affords a more convenient means of ascertaining the storage constant for any stream. The average storage for any draft

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1583, Table 8.

having been computed as already outlined, the diagram is entered with this quantity expressed in terms of average run-off, and at the intersection of the draft and storage lines the storage constant in day's flow may be read.

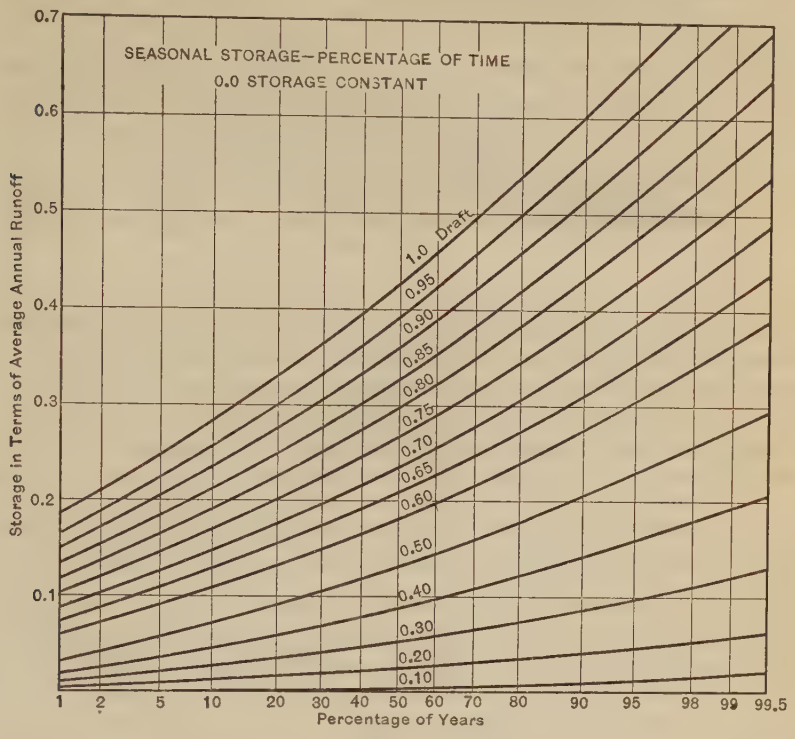

FIG. 6.

Theoretical Inconsistencies of the Seasonal Storage Curves.—When the seasonal storage curves are extended to include drafts that exceed the mean stream flow certain inconsistencies appear, as follows: For the 1.0 draft Hazen adopted an arbitrary quantity of 160 days' mean-year storage, equivalent to 0.438 of the average annual run-off. In Fig. 8 this is represented by *A B*. If the draft is increased the storage must increase also. At some draft, equal to the highest rate of run-off during the year, the seasonal storage required will be equal to the draft minus the annual run-off (Draft—1), and this relation will hold for all higher drafts. That is, the storage curve will eventually become tangent to the straight line, *B C*, which has a slope of 45 degrees. The complete storage curve for 0-day constant will then be the line, *O A C*. For some other stream having a 56-day constant, for example, Hazen's curve will be *O E H*. This is obtained by subtracting 56 days' storage from the 0-day curve, for each draft. This curve, *O E H*, evidently cannot become tangent to the line, *B C*, although theoretically it should do so, as the average annual run-off is still unity as before. If the curves are to be extended so as to apply to drafts in excess of the average run-off, a different formula must be used, and they must become tangent to the line, *B C*, somewhat as shown by the curve, *O E F*.

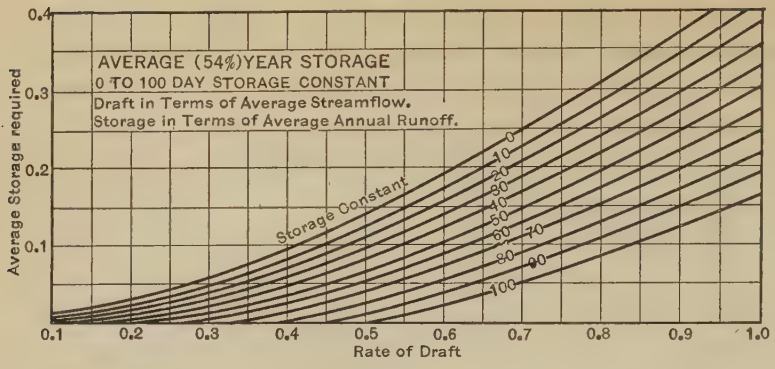

FIG. 7.

Theoretically, a stream might require more storage than is indicated by the 0-day constant. It would then have a negative storage constant. The extreme case would be a stream requiring storage equal to a draft for all rates of draft. For a draft of 1.0, 365 days' storage would be necessary, and the storage constant would become —205 days. The storage curve for such a stream would be the straight line, *O D* (Fig. 8). All storage draft curves

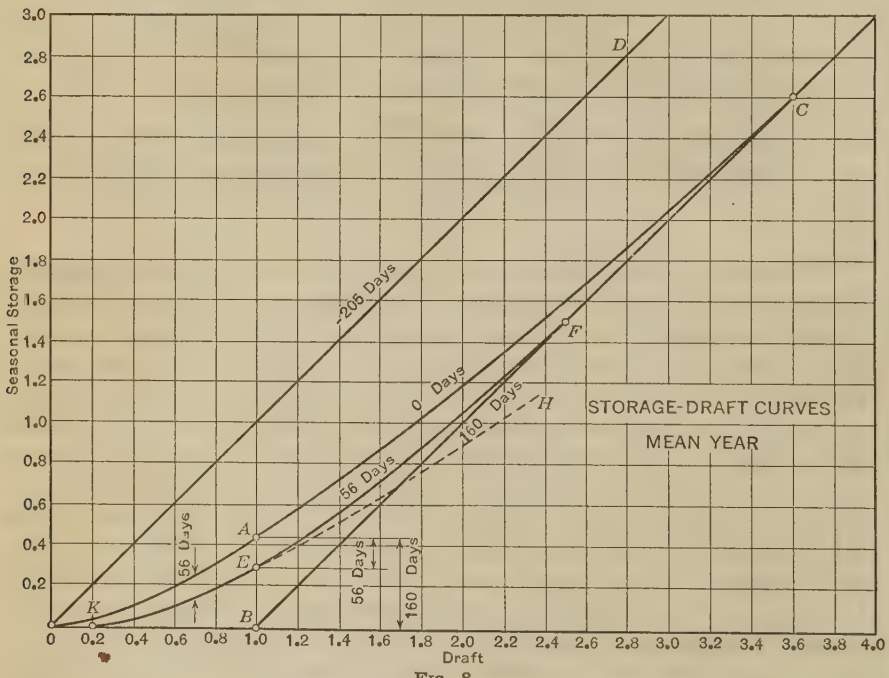

FIG. 8.

theoretically must lie between the limiting curves, *O D* and *O B C*. If the annual run-off in the 95%-year, for example, is 0.8, the seasonal storage for this year will eventually become equal to Draft — 0.8 when the draft becomes large enough.

Combining Seasonal and Annual Storage

Seasonal storage represents the total deficiency of flow of single years. If the flow for the year is less than the draft there will be an annual deficiency. The draft cannot be maintained unless the annual deficiencies are provided for by a reservoir capacity equal to the combined deficiencies of the dryest series of years expected, plus a certain quantity of additional storage which may be used and again replaced year after year. This additional quantity will be given the name "monthly" storage. Its magnitude, to combine with annual storage for various drafts, is to be found.

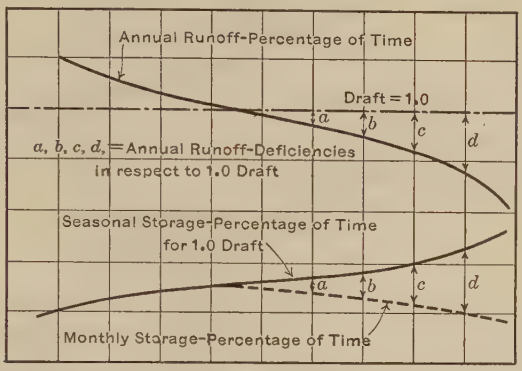

Fig. 9.—Graphic Relations of Annual Seasonal and Monthly Deficiencies.

Examination shows that although the seasonal storage required to maintain a given draft may vary considerably for two yearly periods having the same run-off, nevertheless, as a general principle, the required seasonal storage is least for years of highest run-off, and increases as the annual run-off decreases. Thus, it may be said that the seasonal "storage-percentage of time" curve is made up of two factors: One due to a decreasing annual run-off as a longer percentage of time is considered, and measuring the annual deficiency of run-off; and the other due to what might be called the accidental fluctuations within the dryer season of the year. Since, however, the annual deficiency has been studied separately, and provision made for it by the use of annual storage, it should not again be included as would be necessary were the total storage for a given percentage of time to be assumed as equal to the sum of annual plus the seasonal storage, for that percentage of time.

The "annual storage-percentage of time" and the "seasonal storage-percentage of time" curves are synchronous with the "annual run-off-percentage of time" curve; therefore the annual deficiencies for every percentage of time in respect to any draft may be found from the "annual run-off-percentage of time" curve and their values subtracted from the corresponding ordinates of the "seasonal storage-percentage of time" curve. This will leave a curve representing the values of the "accidental" seasonal deficiencies previously mentioned, called the curve of "monthly" storage. The "annual storage-percentage of time" curve, when superposed on this "monthly storage-percentage of time" curve, indicates the total storage required. Fig. 9 shows an

"annual run-off-percentage of time curve" with deficiencies in respect to a draft of 1.0; and below, a "seasonal storage-percentage of time" curve for this draft, with the several deficiencies to be subtracted, leaving the so-called "monthly" storage curve. This follows the seasonal storage curve up to that percentage of time for which the run-off exactly equals the draft, then breaks and falls below the seasonal storage curve, as shown.

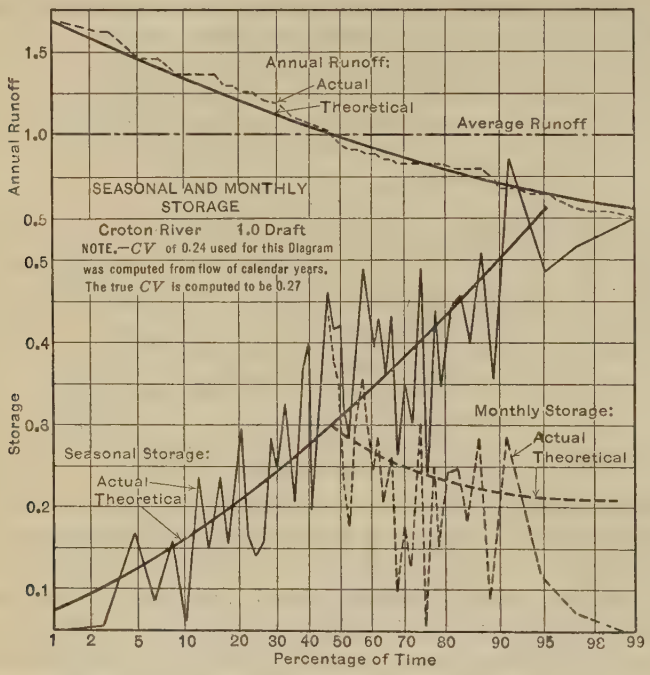

FIG. 10.

Fig. 10 shows the application of the principle to the Croton River. The actual annual run-off for fifty-two years of record is set in descending order of magnitude alongside the theoretical "run-off-percentage of time" curve for such a stream, as measured by its $C\ V$. The actual seasonal storage for years corresponding with the run-off as shown on the upper curve is indicated in Fig. 10, as also are the "monthly" storage values obtained by subtracting the actual annual deficiencies from the seasonal storage. The corresponding theoretical curves are also drawn. The "accidental" fluctuations of seasonal storage for years of nearly the same run-off are clearly evident, and the theoretical curves seem to represent the means of these fluctuations fairly well.

Despite the fact that the "monthly storage-percentage of time" curves break sharply at the percentage of time when the run-off just equals the draft, the curves of total storage as obtained by adding the annual storage to the monthly storage values for identical percentages of time, are fair throughout their length, as should no doubt be the case. Where large draft is considered and the stream has a high $C\ V$, however, the rate of curvature

changes rapidly beyond that percentage of time for which the run-off just equals the draft, as may be seen in Fig. 11.

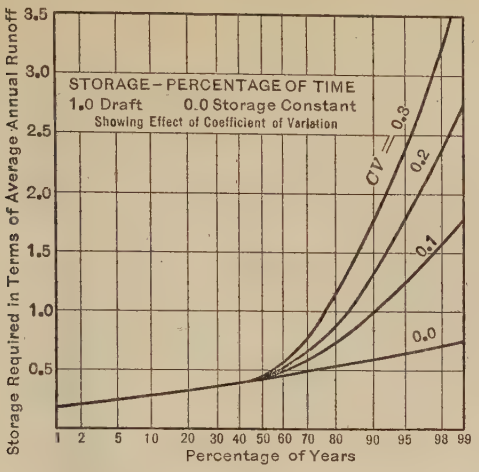

Fig. 11.

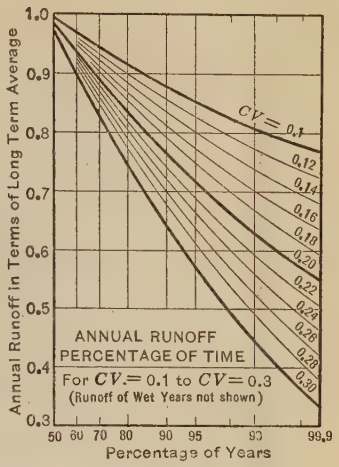

Fig. 12.

Values of annual deficiency, to be subtracted from seasonal storage in order to produce the monthly storage curves, are taken from Fig. 12. Consider, for example, a stream the $C\,V$ of which is 0.3. The run-off at 95% of the time is 0.565; the deficiency in respect to 1.0 draft is, therefore, 0.435, and, in respect to 0.9 draft, 0.335, etc. The seasonal storage for 0.0 constant, 95% of the time, 1.0 draft (Fig. 6) is 0.66, and subtracting the 0.435 annual deficiency from this leaves 0.225 as the 95% monthly storage value.

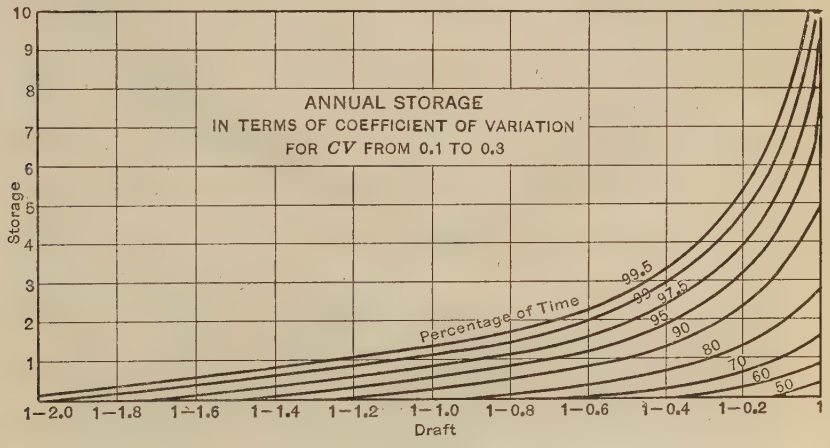

Fig. 13.

The annual storage for $C\,V = 0.3$ and 1.0 draft (Fig. 13) is 7.4 units of $C\,V$, or $7.4 \times 0.3 = 2.22$; and the total storage equals $2.22 + 0.225$, or 2.445 times the average annual run-off of the stream considered.

The complete storage curves as computed for 90%, 95%, and 99% of the time, are shown on Figs. 14, 15, and 16.

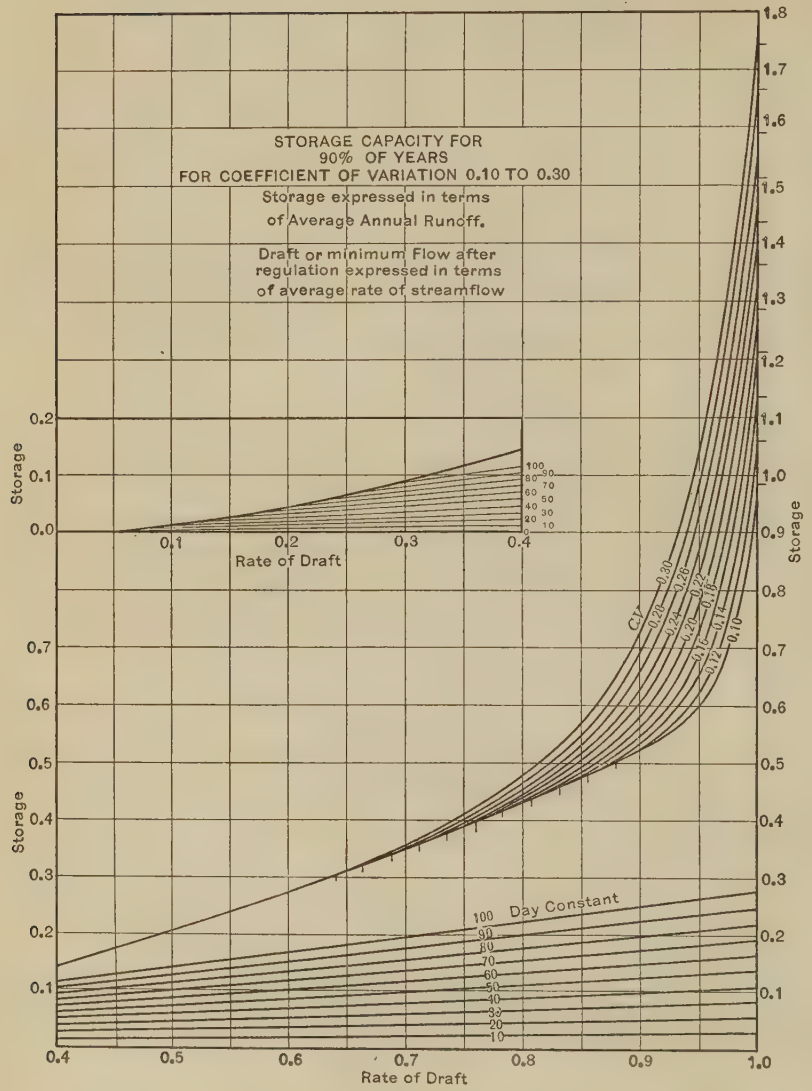
STORAGE CAPACITY FOR
90% OF YEARS
FOR COEFFICIENT OF VARIATION 0.10 TO 0.30
Storage expressed in terms
of Average Annual Runoff.
Draft or minimum Flow after
regulation expressed in terms
of average rate of streamflow
Storage
Rate of Draft
100 Day Constant
C.V.
0.30
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10

FIG. 14.

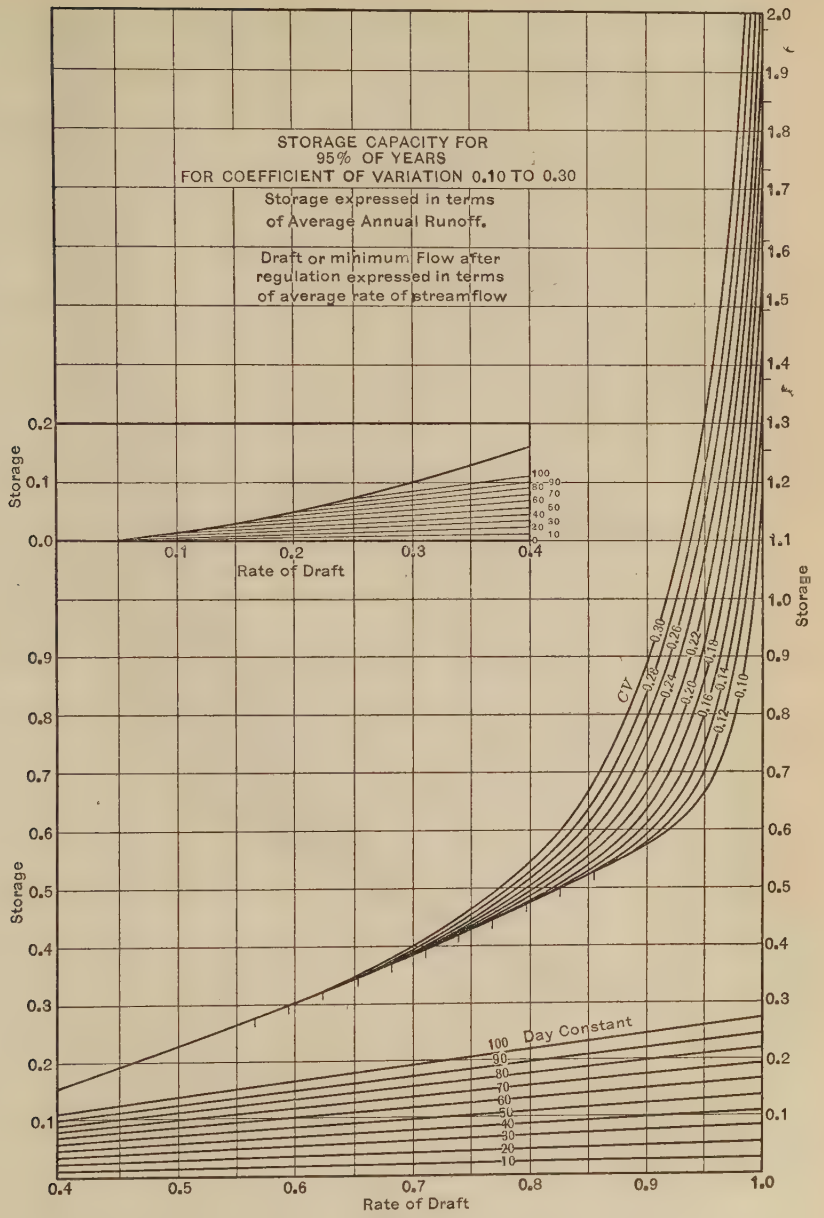
STORAGE CAPACITY FOR
95% OF YEARS
FOR COEFFICIENT OF VARIATION 0.10 TO 0.30
Storage expressed in terms
of Average Annual Runoff.
Draft or minimum Flow after
regulation expressed in terms
of average rate of streamflow
Storage
Rate of Draft
CV
0.30
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
100 Day Constant
90
80
70
60
50
40
30
20
10

FIG. 15.

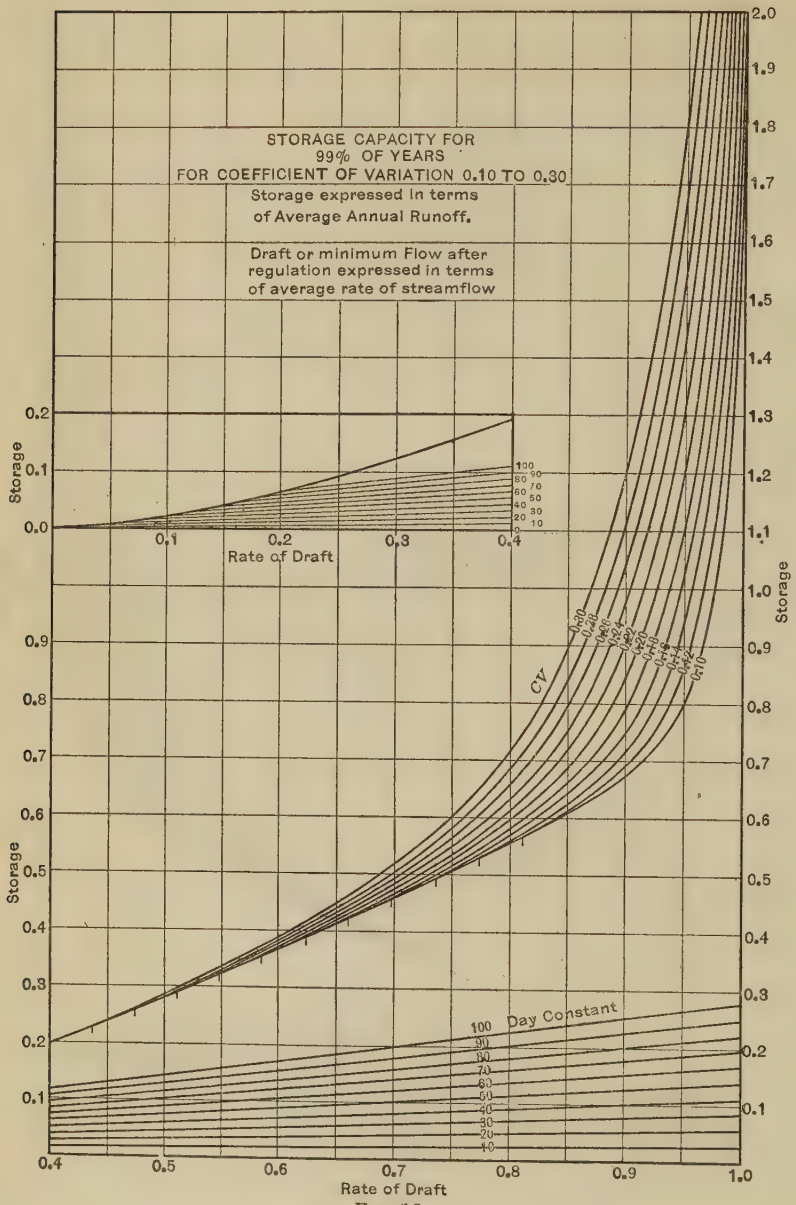
STORAGE CAPACITY FOR
99% OF YEARS
FOR COEFFICIENT OF VARIATION 0.10 TO 0.30
Storage expressed in terms
of Average Annual Runoff.
Draft or minimum Flow after
regulation expressed in terms
of average rate of streamflow
Storage
Rate of Draft
100 90 80 70 60 50 40 30 20 10
CV
0.30
0.25
0.20
0.15
0.10
100 Day Constant
90
80
70
60
50
40
30
20
10

FIG. 16.

Tests of the Storage Curves

To test the dependability of the curves, storage has been taken as indicated for the computed $C\,V$, storage constant, percentage of time, and draft, for four localities with the results shown herewith. In the first two series only annual storage was tested, and in the remainder, total storage.

1.—Annual Storage.—Hudson River at Mechanicville, N. Y.—

Computed $C\,V = 0.16$; 20-year flow record.

1.0 draft: Annual storage for 95% regulation = 1.18 (Fig. 13) as computed.
Result: Draft maintained 100% of time.

1.0 draft: Annual storage for 90% regulation = 0.80 as computed.
Result: Draft maintained 17 years, or 85% of time, with deficiencies of 0.01, 0.08, and 0.03, in the years 1917, 1918, and 1919, respectively.

0.9 draft: Annual storage for 95% regulation = 0.176 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.074 in 1911 and 0.074 in 1915.

0.9 draft: Annual storage for 90% regulation = 0.115 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.135 in 1911 and 0.135 in 1915.

0.8 draft: Annual storage for 95% regulation = 0.32 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.108 in 1911 and 0.118 in 1915.

2.—Annual Storage.—Croton River, New York.—

Computed $C\,V = 0.25$; 52-year record.

1.0 draft: Annual storage for 95% regulation = 1.85 (Fig. 13) as computed.
Result: Draft maintained 48 years, or 92.4% of time, with deficiencies of 0.16 in 1883, 0.17 in 1884, 0.10 in 1885, and 0.16 in 1886.

1.0 draft: Annual storage for 90% regulation = 1.24 as computed.
Result: Draft is maintained 45 years, or 86.4% of time, with deficiencies of 0.17 in 1880, 0.21 in 1881, 0.17 in 1882, 0.22 in 1883, 0.17 in 1884, 0.10 in 1885, and 0.16 in 1886.

0.9 draft: Annual storage for 95% regulation = 0.45 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.08 in 1881, 0.07 in 1882, 0.12 in 1883, 0.07 in 1884, and 0.06 in 1886.

0.9 draft: Annual storage for 90% regulation = 0.33 as computed.
Result: Draft maintained 44 years, or 84.6% of time, with deficiencies of 0.07 in 1870, 0.08 in 1871, 0.09 in 1880, 0.11 in 1881, 0.07 in 1882, 0.12 in 1883, 0.07 in 1884, and 0.06 in 1886.

0.8 draft: Annual storage for 95% regulation = 0.20 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.11 in 1870, 0.02 in 1871, 0.01 in 1880, 0.01 in 1881, and 0.04 in 1910.

0.8 draft: Annual storage for 90% regulation = 0.11 as computed.
Result: draft maintained 45 years, or 86.5% of time, with deficiencies of 0.20 in 1870, 0.02 in 1871, 0.03 in 1876, 0.10 in 1880, 0.01 in 1881, and 0.07 in 1891.

3.—Total Storage.—Hudson River at Mechanicville, N. Y.—

$C\,V = 0.16$; 60-day storage constant; 20-year weekly flow record.

1.0 draft: Storage for 95% regulation = 1.45 as computed.
Result: Draft maintained 20 years, or 100% of time. Storage of 1.33 actually needed.

0.95 draft: Storage for 95% regulation = 0.65 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.02 and 0.07 in 1918 and 1919, respectively.
0.95 draft: Storage for 90% regulation = 0.53 as computed.
Result: Draft maintained 16 years, or 80% of time, with deficiencies of 0.01 in 1911, 0.12 in 1912, 0.02 in 1917, and 0.07 in 1919.
0.9 draft: Storage for 95% regulation = 0.46 as computed.
Result: Draft maintained 19 years, or 95% of time, with deficiencies of 0.03 in 1911.
0.9 draft: Storage for 90% regulation = 0.40 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.02 in 1910 and 0.07 in 1911.
0.8 draft: Storage for 95% regulation = 0.35 as computed.
Result: Draft maintained 20 years, or 100% of time, but actual storage of 0.33 is needed in 1908, 1909, 1911, and 1914.
0.8 draft: Storage for 90% regulation = 0.30 as computed.
Result: Draft maintained 17 years, or 85% of time, with deficiencies of 0.03 in 1908, 0.03 in 1909, and 0.01 in 1910.

4.—Total Storage.—Croton River, New York.—

CV = 0.25; 33-day storage constant; 52-year monthly record.
1.0 draft: Storage for 95% regulation = 2.06 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.24 in 1883, 0.07 in 1884, 0.22 in 1885, and 0.14 in 1886.
1.0 draft: Storage for 90% regulation = 1.47 as computed.
Result: Draft maintained 45 years, or 86.4% of time, with deficiencies of 0.24 in 1880, 0.16 in 1881, 0.02 in 1882, 0.41 in 1883, 0.07 in 1884, 0.22 in 1885, and 0.14 in 1886.
0.95 draft: Storage for 95% regulation = 1.025 as computed.
Result: Draft maintained 46 years, or 88.5% of time, with deficiencies of 0.16 in 1880, 0.12 in 1881, 0.33 in 1883, 0.02 in 1884, 0.17 in 1885, and 0.08 in 1886.
0.95 draft: Storage for 90% regulation = 0.82 as computed.
Result: Draft maintained 45 years, or 86.4% of time, with deficiencies of 0.07 in 1872, 0.30 in 1880, 0.12 in 1881, 0.33 in 1883, 0.02 in 1884, 0.17 in 1885, and 0.08 in 1886.
0.9 draft: Storage for 95% regulation = 0.69 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.02, 0.05, 0.22, and 0.11.
0.8 draft: Storage for 95% regulation = 0.44 as computed.
Result: Draft maintained 48 years, or 92.3% of time, with deficiencies of 0.01, 0.02, 0.11, and 0.01.
0.7 draft: Storage for 95% regulation = 0.33 as computed.
Result: Draft maintained 51 years, or 98% of time, with deficiency of 0.06 in 1880.

The deficiencies after regulation are due, in the cases of the higher drafts, to groups of dry years. The flow of the Croton River, for example, was low continuously from 1879 to 1886. For lower drafts the deficiencies usually occur one or two years at a time, and are due to seasonal fluctuation as well as to low annual run-off.

Examination of the tests seems to indicate that even a 20-year flow record is not sufficient in itself to provide consistent, reasonable results when various drafts and percentages of time are considered. Thus, the theoretical annual storage for 95% regulation of the Hudson River for 1.0 draft maintains the flow 100% of the time, while the 90% storage maintains it only 85% of the

time. The annual storage for 90% regulation, 0.9 draft, is also sufficient for 95% of time. There seems to be no assurance that the grouping of dry years in one 20-year record will be similar, in respect to storage requirements, to that of the next 20 years.

The storage curves, Figs. 14, 15, and 16, are designed as suitable for streams having CV of 0.30 or less. For larger coefficients the annual storage computed from Fig. 5 will be too small, since the coefficient of skew should be greater, corresponding to more years of deficient run-off. However, a test has been made of the annual storage required for the Murray River, at Mildura, Australia, having a computed CV of 0.53, as based on a run-off of 48 calendar years. The theoretical annual storage needed, as taken from Fig. 5, is as follows:

1.0 draft.....	95%	annual storage	= 7.39 CV	= 3.92	average run-off
" "	90%	" "	= 4.97 CV	= 2.64	" "
" "	80%	" "	= 2.80 CV	= 1.48	" "

Using these computed storages it was found that the theoretical 95% storage maintained the draft for 92% of the years; the 90% storage, for 85% of the years; and the 80% storage, for 75% of the years.

The annual run-off of this river frequently reaches 175 to 200% of its average, or falls as low as 55% of its average, while in 1903 the run-off fell to 0.16 of its average. The computed coefficient of skew (CS) is 1.48, whereas the storage curves used were based on a CS of 0.6.

Percentage of Years in Which Partial Depletion Occurs

In Fig. 17 an "annual run-off-percentage of time" curve is drawn. Assuming no regulation by the use of storage, a nominal draft or installed wheel capacity equivalent to 0.947 mean stream flow could operate at full capacity for 60% of the time, after which the rate of flow becomes deficient, dropping to 0.0 at 100% of the time, as shown. Computing the area below the draft line and the run-off curve it is found that with installed capacity of 0.947, an average use of 0.907 could be obtained, and the natural flow would exceed this average during 72% of the time. The deficiencies in respect to this average use are measured by the area, b, which is equal to the area of excesses, a.

Computing thus the average use available for various installations, or "nominal drafts", a curve may be drawn as in Fig. 18. In this diagram the nominal draft curve is a "run-off-percentage of time" curve, and drafts are expressed in terms of CV. Reading from this diagram, for example, a draft of $1 - 0.33\,CV$ may be maintained 60% of the time, and the average use available to an installation equivalent to this draft, is $1 - 0.57CV$, found at the intersection of the lower curve with the 60% ordinate. Following this average use of $1 - 0.57\ CV$ horizontally to the right, its intersection with the run-off or nominal draft curve occurs at 70% of the time, indicating that this average use may be exceeded during 70% of the years. If the stream being considered has $CV = 0.16$, the nominal draft of $1 - 0.33\ CV = 1 - (0.33 \times 0.16) = 0.947$, and the average use $= 0.907$, as in Fig. 17.

If, however, it is a minimum use of 0.907 that is required, water must be stored, equal to the deficiencies measured by the area, *b*, Fig. 17, and this must be taken from flow that exceeds the nominal draft or installed capacity. It is reasoned that as a general condition this storage probably will be taken about as represented by the area, *a*, which equals *b*. If this is true, then with complete or 100% regulation, Fig. 17 would show that with a draft or capacity of 0.907, there will be 60 years during which the run-off is so high that the reservoir is completely filled and waste flow occurs; 10 years (between 60 and 70% of the time) during which the run-off exceeds the draft, but not sufficiently to make good the entire depletion; and 30 years (between 70 and 100%) during which the run-off is deficient in respect to the draft, and storage must be drawn upon.

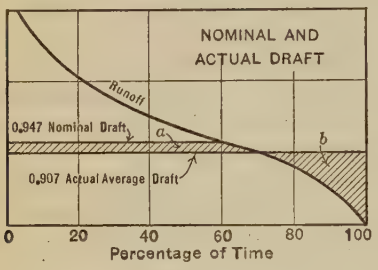

FIG. 17.

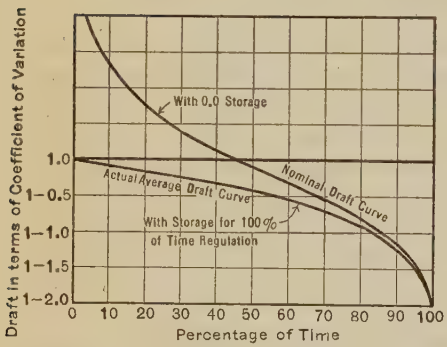

FIG. 18.—PERCENTAGE OF YEARS RESERVOIR WILL BE COMPLETELY FULL.

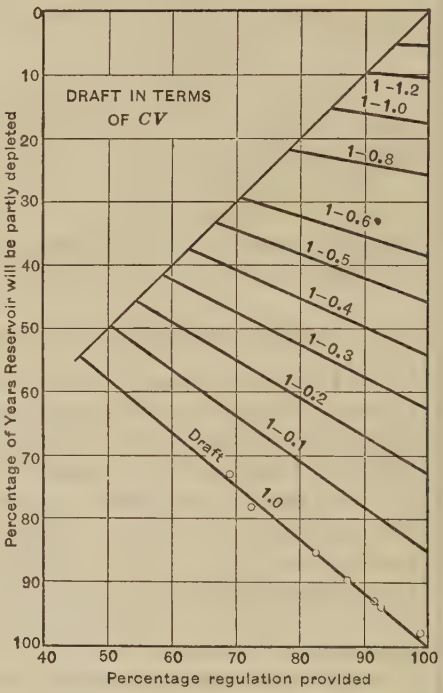

FIG. 19.—PERCENTAGE OF YEARS PARTIAL DEPLETION OCCURS.

Following this argument the two curves of Fig. 18 represent conditions with no annual storage and with storage for 100% of the time. Thus, for 1.0 draft (equal to the average stream flow) with complete regulation, the reservoir will be exactly full 1 year in 100, or 1 000, or whatever period is assumed as 100% of the time, and partial depletion will continue for the remainder of the time. With no storage, the reservoir (of 0.0 capacity) will be full during the 46 wet years in each 100, and partly depleted during the remaining 54 years. The intersection with the two curves of a horizontal line representing any draft shows the percentage of time the reservoir will be full with complete storage and with no storage.

For intermediate conditions, as, for example, when regulation for 90% of the time is provided, Fig. 19 indicates the relations. This diagram is formed by replotting the values from the curves of Fig. 18, and connecting them by straight lines. As an example of its use assume a stream having a $CV = 0.2$, and a draft equal to 0.9 of the mean stream flow, or $1 - 0.5\ CV$. The diagram shows that this draft would be maintained 67% of the years without any storage. Now, if storage is provided to regulate for 90% of the time, the reservoir will be in a partly depleted condition during 42% of the years.

Although perhaps Fig. 19 is not entirely conclusive, it does take into account the most severe condition of draft. As a check on its construction an examination of the 1 000-year artificial flow record was made with storages sufficient to maintain regulation for various percentages of time, for a draft equal to the mean flow, or 1.0. The results are shown by the small circles on Fig. 19, which nearly agree with the corresponding draft line.

Continuous Periods of Partial Depletion

To ascertain the probable longest periods during which a reservoir will be partly depleted, an examination of the 1 000-year artificial record was made, using various rates of draft with appropriate storage for 90% of the time, and, again, for 95% of the time. The lengths of periods between which the reservoir refilled when so operated, were tabulated, and the averages of the ten longest periods in each instance were taken as the probable lengths of the longest continuous periods to be expected once in 100 years. The results are shown on Fig. 20. Interpolating for a stream such as the Hudson River, the periods are as given in Table 2.

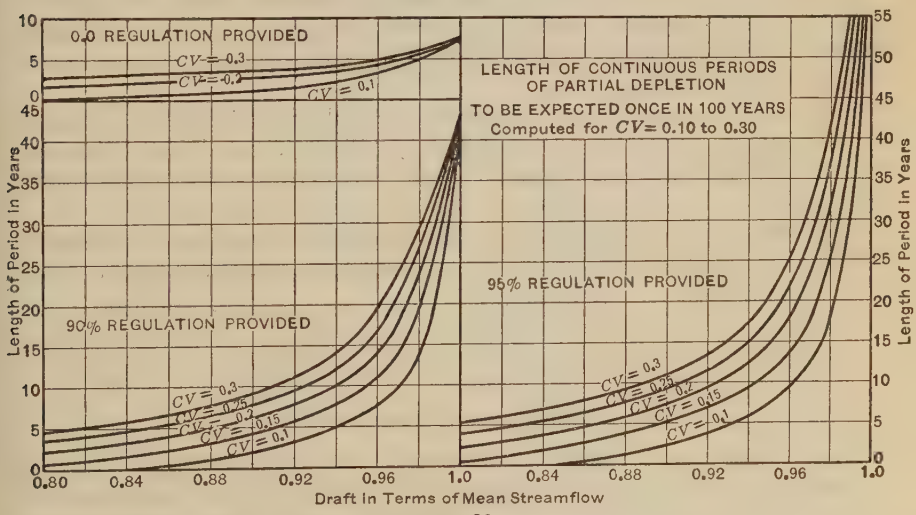

Fig. 20.

Where no annual storage is provided Fig. 20 simply indicates the probable longest periods during which the annual run-off will be continuously less

than an assumed draft. For the Hudson River, during a period of about 5 years once in 100 years, the run-off will be continuously less than 0.98; and for a 3-year period, it will be less than 0.90.

TABLE 2.—CONTINUOUS PERIODS OF PARTIAL DEPLETION FOR HUDSON RIVER ($CV = 0.16$).

Assumed draft, (mean stream flow).	Percentage of regulation.	Length of depletion period, in years.
0.98	90	21
0.98	95	27
0.95	90	10
0.95	95	12
0.90	90	5
0.90	95	6

COMPUTING THE COEFFICIENT OF VARIATION (CV)

In computing CV the annual run-off of calendar years, or some other 12-month period, as, for example, the water-year beginning in October or November, may be used; but it is believed that the most satisfactory results may be obtained by examining the years solely in respect to storage conditions. Natural stream flow appears on a mass curve generally in wave form, with crests approximately but not uniformly 12 months apart. From the viewpoint of regulation, the year begins following the crest at the annual flood period, when the rate of flow drops below the mean intensity. Thus, each year starts when depletion of the storage begins, continues on through the dryer season, and the following wet season, and ends when the reservoir has been refilled as much as will be possible that year, and depletion again begins. Considered in this way the year may be as short as 9 months, or as long as 14 or 15 months.

The CV thus computed will be larger as a rule than for uniform periods of 12 months. Thus, the CV of the Croton River as based on 52 calendar years is 0.238, while using "storage" years, if that expression is permissible, it is 0.25.

The possible error in the CV of the Croton River, as computed from a short record, is suggested by the following tabulation, based on calendar years:

52-year period.....CV ..... = 0.238
20-year periods....CV ranges from 0.226 to 0.276; ave. = 0.251
10-year periods....CV " " 0.157 " 0.320; " = 0.243
5-year periods....CV " " 0.049 " 0.405; " = 0.245

STORAGE ON A TRIBUTARY

Not infrequently a reservoir is located some distance above the point of use, or on a branch stream, providing a condition wherein the average run-off at the power site or point of use considerably exceeds the average run-off tributary to the reservoir. In such instances, as part of the stream flow that reaches the point of use does not pass through the reservoir, a question arises as to the usefulness of the storage.

As the position of the reservoir does not affect the natural flow at any point, the amount of storage which must be released during dry seasons is identical for all reservoir locations; therefore, the limiting condition lies in the ability of the catchment area tributary to the reservoir to supply the storage which must be released.

For the purpose of this discussion the whole run-off or stream flow passing the point of use will be assumed as unity; the portion that is tributary to the reservoir, and therefore may be controlled, will be called the "tributary" run-off, and its proportion to the whole will be called the "tributary ratio"; and the portion passing the point of use without going through the reservoir will be called "main-stream" portion, and its proportion to the whole will be called the "main-stream ratio". In Fig. 21, the run-off at *A*, the point of use, corresponds to a catchment area of 100 sq. miles, of which 60 sq. miles is tributary to the reservoir, *B*. The "tributary ratio" is, therefore, $\frac{60}{100}$, or 0.6, while the "main-stream ratio" is $\frac{40}{100}$, or 0.4.

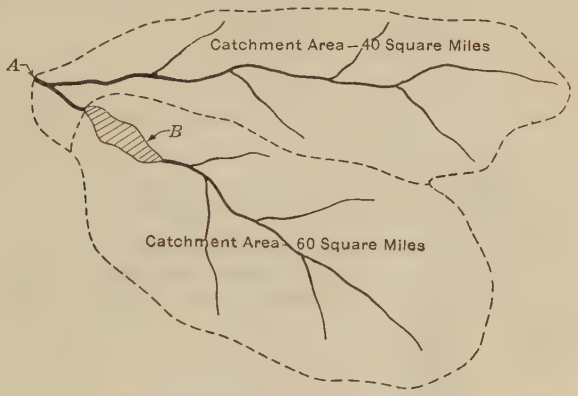

FIG. 21.—ILLUSTRATION OF STORAGE ON A TRIBUTARY.

Flow that passes into the reservoir may be controlled; but when this flow is completely shut off while the flow of the main stream passes the point of use at rates exceeding the assumed draft, the excess is lost to use. On this excess depends the usefulness of the storage, so an effort will be made to find its value.

A "daily run-off-percentage of time" curve embracing a considerable period of years may be assumed to give a good approximation of the relations between various rates of flow and the periods during which such flow is exceeded. As the area enclosed below such a curve measures the average run-off, the areas above lines representing various rates of flow or draft measure the average quantities of excess, referred to these drafts. From study of a number of "daily run-off-percentage of time" curves of Eastern streams for which the storage constants had been computed, a set of theoretical curves were drawn (Fig. 22). Presumably these curves would not fit all streams having

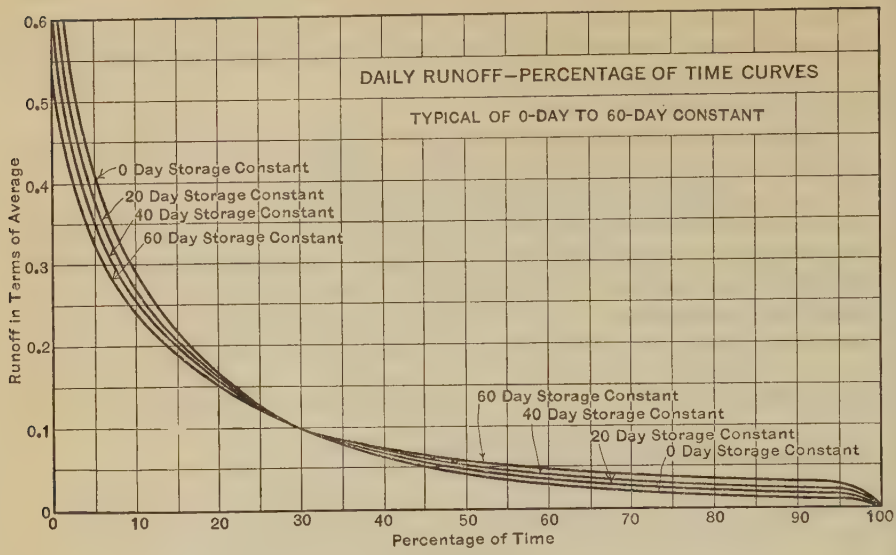
DAILY RUNOFF—PERCENTAGE OF TIME CURVES
TYPICAL OF 0-DAY TO 60-DAY CONSTANT
0 Day Storage Constant
20 Day Storage Constant
40 Day Storage Constant
60 Day Storage Constant
60 Day Storage Constant
40 Day Storage Constant
20 Day Storage Constant
0 Day Storage Constant
Runoff in Terms of Average
Percentage of Time
0.6
0.5
0.4
0.3
0.2
0.1
0
0
10
20
30
40
50
60
70
80
90
100

FIG. 22.

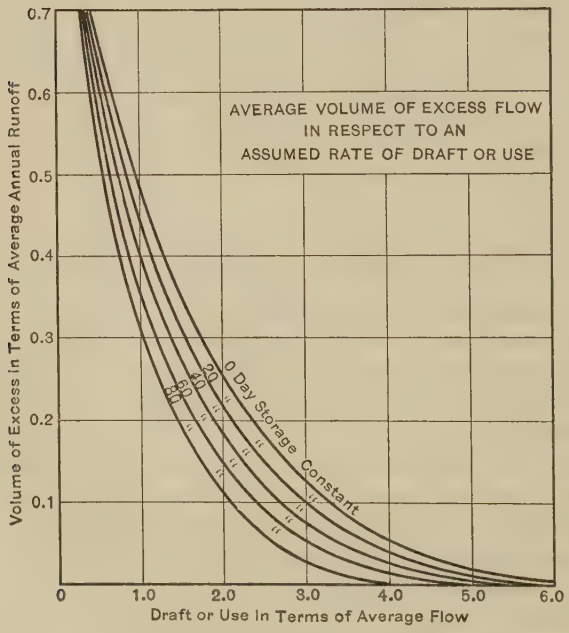
AVERAGE VOLUME OF EXCESS FLOW
IN RESPECT TO AN
ASSUMED RATE OF DRAFT OR USE
0 Day Storage Constant
20
40
60
Volume of Excess in Terms of Average Annual Runoff
Draft or Use in Terms of Average Flow
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
1.0
2.0
3.0
4.0
5.0
6.0

FIG. 23.

similar storage constants, but would approximate them and show that an increase in the storage constant corresponds to decreased flood flow and increased dry-season flow. By measuring the areas enclosed above various rates of draft in Fig. 22, the curves of Fig. 23 were drawn, indicating the approximate volume flowing in excess of any assumed rate. For example, if an installation were provided equivalent to 0.8, the average unregulated flow of a stream having a 60-day storage constant, the diagram shows that 42.5% of the average annual run-off would pass at rates in excess of this installation, thus being lost to use.

Maximum Draft.—With a 100% tributary ratio or storage at the point of use, the maximum possible draft will equal the long-term average stream flow. With some smaller tributary ratio, indicating that a part of the run-off does not pass through the reservoir, the maximum possible draft, even with unlimited storage capacity, will be equal to the average flow of the tributary area, plus a portion of the main stream flow depending on the fluctuations of this stream. If too large a regulated flow or draft is assumed, the water flowing into the reservoir will be insufficient to maintain the regulation. The maximum drafts for various tributary ratios and storage constants are computed from Fig. 23, as follows: Various drafts at the point of use are assumed, expressed in terms of the average stream flow of the whole area. Dividing these drafts by the main-stream ratio reduces them to terms of average flow of the main stream portion. Excess flow, or water lost in relation to these drafts, is taken from Fig. 23 and multiplied by the main-stream ratio to reduce it to terms of the whole average flow. These excesses in relation to various drafts, subtracted from 1.0, the average flow of the whole area, show the useful flow available for various drafts. For each proportion of tributary ratio and each storage constant, there is some particular draft that will exactly equal the average water available for that draft. This is the maximum possible draft. Computations based on Fig. 23 for storage constants of 0 to 100 days, are shown on Fig. 24.

Maximum Draft without Annual Storage.—Storage on a tributary is just as useful as storage at the point of use as long as the available flow in the dryest year considered is equal to the assumed draft; for while this condition obtains, the reservoir can be refilled each year. When the available flow of the dryest year is less than the annual draft, annual storage will be required. The limiting draft without annual storage is assumed to bear the same relation to the maximum possible draft as the dryest year's run-off bears to the average year's run-off. This assumption is made on the theory that the characteristic "run-off-percentage of time" curve for any dry year is of the same type as that of the average year, as drawn in Fig. 22, every ordinate of the dry-year curve being reduced however, in the proportion indicated by the CV of the stream considered. If this is true, in the dry year both the draft and the water available to that draft are reduced in the same proportion, and as all the available water is used in the average year, so also will it all be used in the dry year.

Seasonal Storage.—As long as the assumed draft does not necessitate the use of annual storage, the conditions for all tributary ratios are the same as for storage at the point of use; therefore, within these limits the required storage may be taken from the seasonal storage curves, Fig. 6.

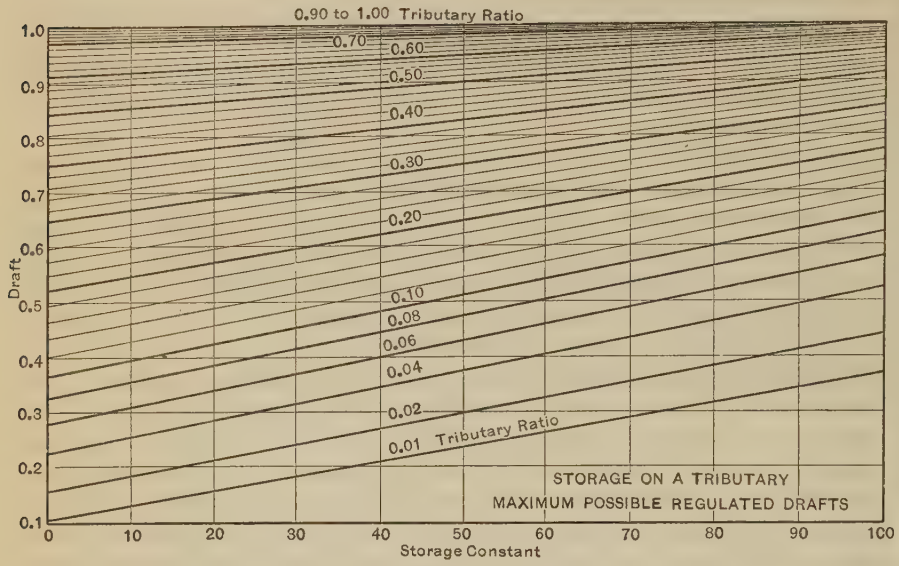

FIG. 24.

Monthly Storage.—With storage at the point of use the required total is assumed to be equal to the seasonal storage minus the annual deficiency plus the annual storage, all taken for the particular type of run-off year considered. In this discussion the annual deficiency in the 95% or other dry year, in respect to the maximum possible draft for any tributary ratio, is assumed to be the difference between the maximum possible draft (which represents the water required), and the limiting draft without annual storage, for the 95% or other type year under examination (which represents the water available in this dry year). For any lower draft the annual deficiency is assumed to be equal to the deficiency in respect to the maximum draft, minus the difference between the two drafts considered.

For example, assume $CV = 0.10$, 0-day storage constant, 0.4 tributary ratio, 95% regulation: The maximum possible draft (Fig. 24) is 0.748; the limiting draft without annual storage is $0.748 \times 0.855^* = 0.640$. The 95% year annual deficiency is $0.748 - 0.640 = 0.108$. For any lower draft, as 0.69, for example, the annual deficiency is assumed as 0.108 minus the difference between the two drafts, or, Draft 0.748 — Draft 0.690 = 0.058; and 0.108 — 0.058 = 0.050 is the annual deficiency for Draft 0.69. The seasonal storage (Fig. 6) for the 0.748 draft = 0.428, and subtracting from this the 0.108 annual deficiency, the remainder or monthly storage is found to be

* This is the 95% run-off for $CV = 0.10$ (Fig. 12).

0.320. The monthly storage for 0.69 and other drafts would be computed in the same manner.

This method is not entirely consistent with the assumption that the flow for every percentage of time for a dry year was a fixed fraction of that for an average year. The annual deficiency for some dry year, in respect to the maximum possible draft, actually would be less than that shown by the method just discussed, because in a year of any given type the water available for various drafts is not a fixed quantity, but increases as the draft increases, although not in the same proportion. The error involved appears to be only about 3% of the average annual run-off in the example just cited, for which only 0.4 of the whole run-off is tributary to the reservoir, and for which the maximum possible draft of 0.748 was assumed. The required storage is nearly equal to the entire average year's run-off. As a lower draft or a larger tributary ratio is considered, the difference is gradually eliminated. Monthly storage could be computed by plotting curves of excess flows in respect to various storage constants and coefficients of variation for 95% or other dry years, and thus ascertaining the excesses for various tributary ratios and drafts; but a large amount of labor would be involved, perhaps without attaining any greater accuracy.

Annual Storage.—The quantities of annual storage required range from 0.0 in the case of limiting draft without annual storage, to a maximum for the maximum possible draft. According to the methods previously discussed the range of drafts requiring annual storage, for each type of stream as indicated by its CV, always embraces the same fraction of the maximum possible draft. For example, again assuming 95% regulation, $CV = 0.10$, 0-day constant: The 95% year run-off (Fig. 12) is 0.855; the maximum possible draft for a 0.4 tributary ratio (Fig. 24) is 0.748; and the limiting draft without annual storage is $0.748 \times 0.855 = 0.640$. Now, for a 0.1 tributary ratio the maximum possible draft (Fig. 24) is 0.365, and, therefore, the limiting draft without annual storage is $0.365 \times 0.855 = 0.312$.

It is now assumed that the annual storage in the case of storage at the point of use, or 100% tributary ratio, depends on the relative drafts considered, so that if the relative drafts are expressed in terms of maximum possible draft the storage for any tributary ratio can be found as the product of the annual storage for the given draft in terms of the maximum possible draft, as required for a 100% tributary ratio, by the maximum possible draft for the given tributary ratio. For example, with $CV = 0.10$ and 0-day storage constant: The maximum draft for a 100% tributary ratio (Fig. 24) is 1.0 and requires 95% year annual storage (Fig. 13) of 0.739. For a 0.9 draft an annual storage of 0.05 is required. Now, with a 0.4 tributary ratio, the maximum possible draft is, as noted, 0.748 and since this represents 100% of the average available run-off the required annual storage is computed as $0.748 \times 0.739 = 0.553$; and a draft of 0.9 maximum, or $0.9 \times 0.748 = 0.673$, requires annual storage of $0.748 \times 0.05 = 0.037$.

If the assumption is true—and it appears not unreasonable—that the flow for every typical year follows the same characteristics as those indicated by

the long-term "run-off-percentage of time" curve, then, in any dry year, considering the main stream flow in respect to the maximum possible draft, actually there will be more water available than this method assumes, as was explained under "Monthly Storage." For example, assuming a 0.5 tributary ratio, a 0-day constant, $CV = 0.16$: The maximum possible draft (Fig. 24) is 0.84, and the water available in the 95% year is taken as the product of 0.84 by the ratio of the 95% year run-off to the average year run-off, which (Fig. 12) is 0.768. The water available in the 95% year is then $0.84 \times 0.768 = 0.645$, leaving an annual deficiency of $0.84 - 0.645 = 0.195$. Now, if a "run-off-percentage of time" curve for the 95% year is drawn on the assumption that the flow at every point is 0.768 as large as that of the average year, it is found that the flow available for the 0.84 draft is 0.680, or 0.035 more than that found by the adopted method. It appears possible, therefore, that the methods herein proposed may provide somewhat too much annual storage, but also somewhat too little monthly storage. Thus, these tend to balance each other. In both cases the errors are largest for the smaller tributary ratios and maximum possible drafts, and are completely eliminated as the tributary ratio increases and the draft decreases.

Total Storage.—Monthly and annual storage are similarly computed for the type of year representing the degree of regulation desired. In the examples already cited and in the remaining diagrams (Figs. 25 to 33) 95% regulation is assumed. The required total storage is equal to the sum of the monthly and the annual storage.

Summary of Computations

1.—The maximum possible draft with unlimited storage (Fig. 24) is affected by the storage constant and the tributary ratio, and is determined from theoretical "run-off-percentage of time" curves (Fig. 22) which are computed to show the characteristic daily fluctuations typical of various storage constants.

The maximum possible draft is that draft in respect to which the average water available is just sufficient. It is not affected by the percentage of time the regulation is to be maintained.

2.—The limiting draft without annual storage is affected by the storage constant, the CV of the annual flow, the tributary ratio, and the percentage of time that regulation is to be maintained.

The limiting draft is the product of the maximum possible draft by the ratio of the run-off of the dryest year considered to the run-off of the average year.

3.—The annual deficiency is affected by the storage constant, the CV, the tributary ratio, and the percentage of time that regulation is to be maintained.

For the maximum possible draft the annual deficiency equals the maximum possible draft (Paragraph 1) minus the limiting draft (Paragraph 2). For any lower draft it equals this quantity diminished by the difference between the maximum possible draft and the given draft.

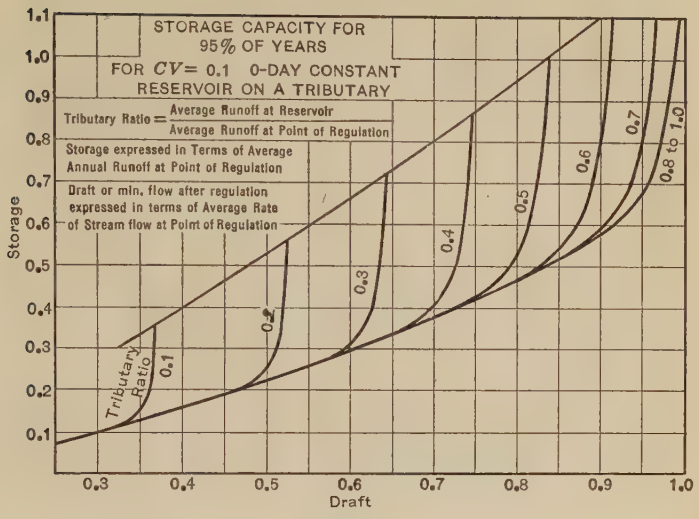
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.1 0-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0

FIG. 25.

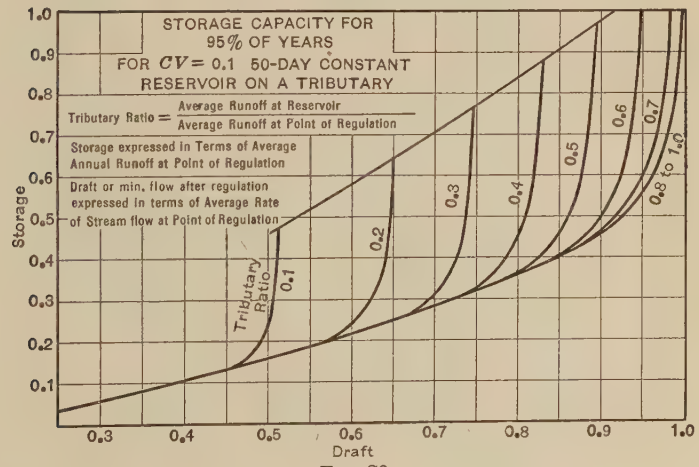
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.1 50-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0

FIG. 26.

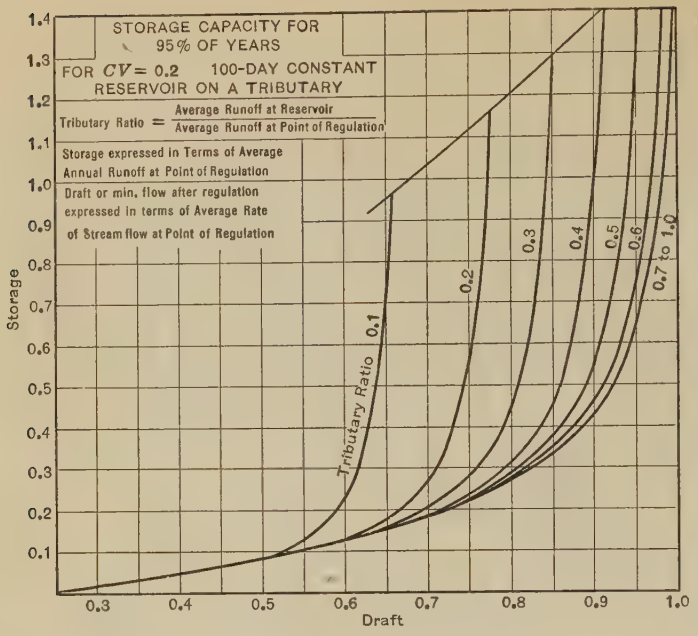
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.2 100-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7 to 1.0

FIG. 27.

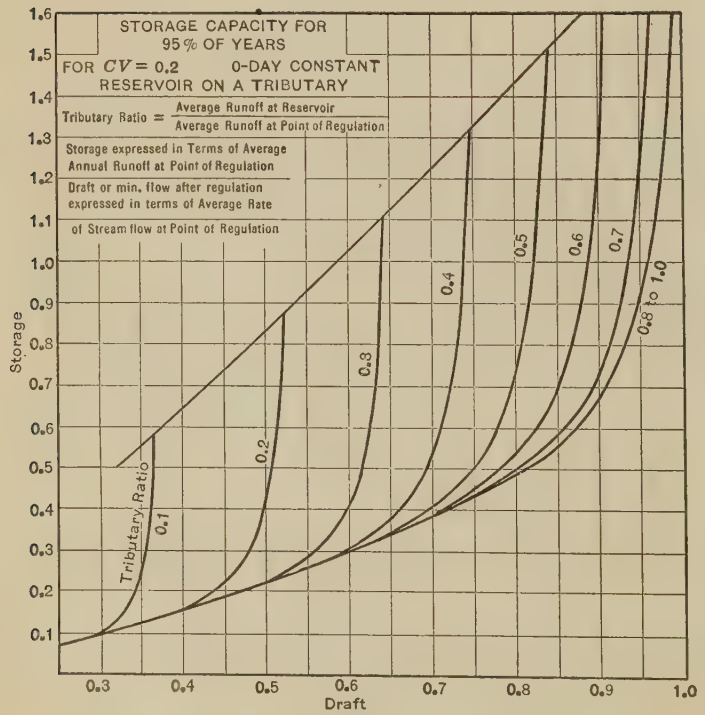
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.2 0-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0

FIG. 28.

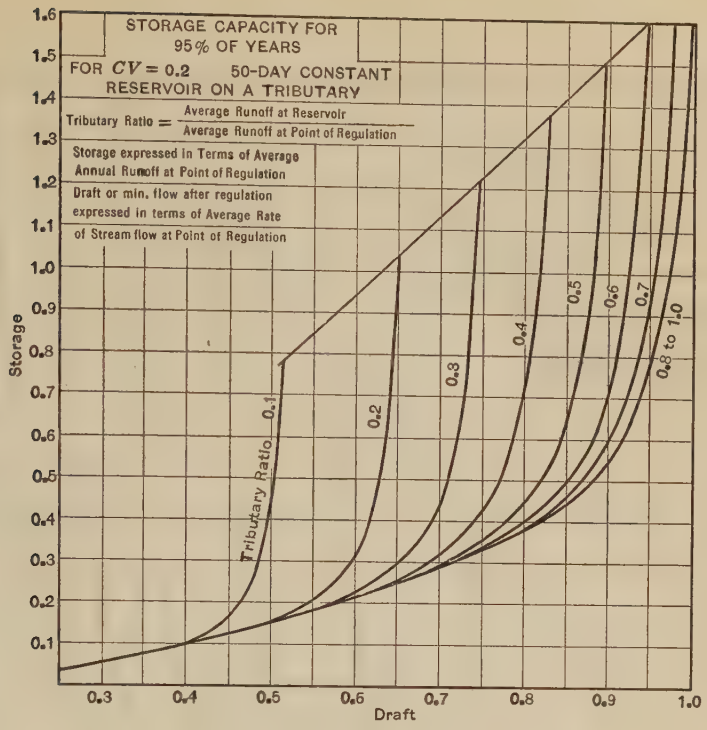
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.2 50-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0

FIG. 29.

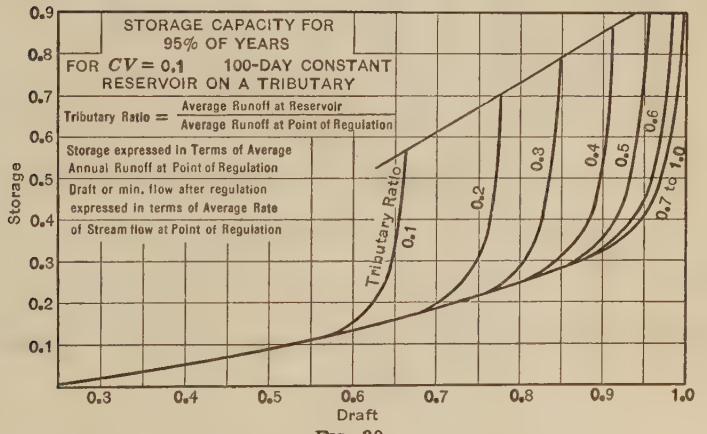
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.1 100-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7 to 1.0

FIG. 30.

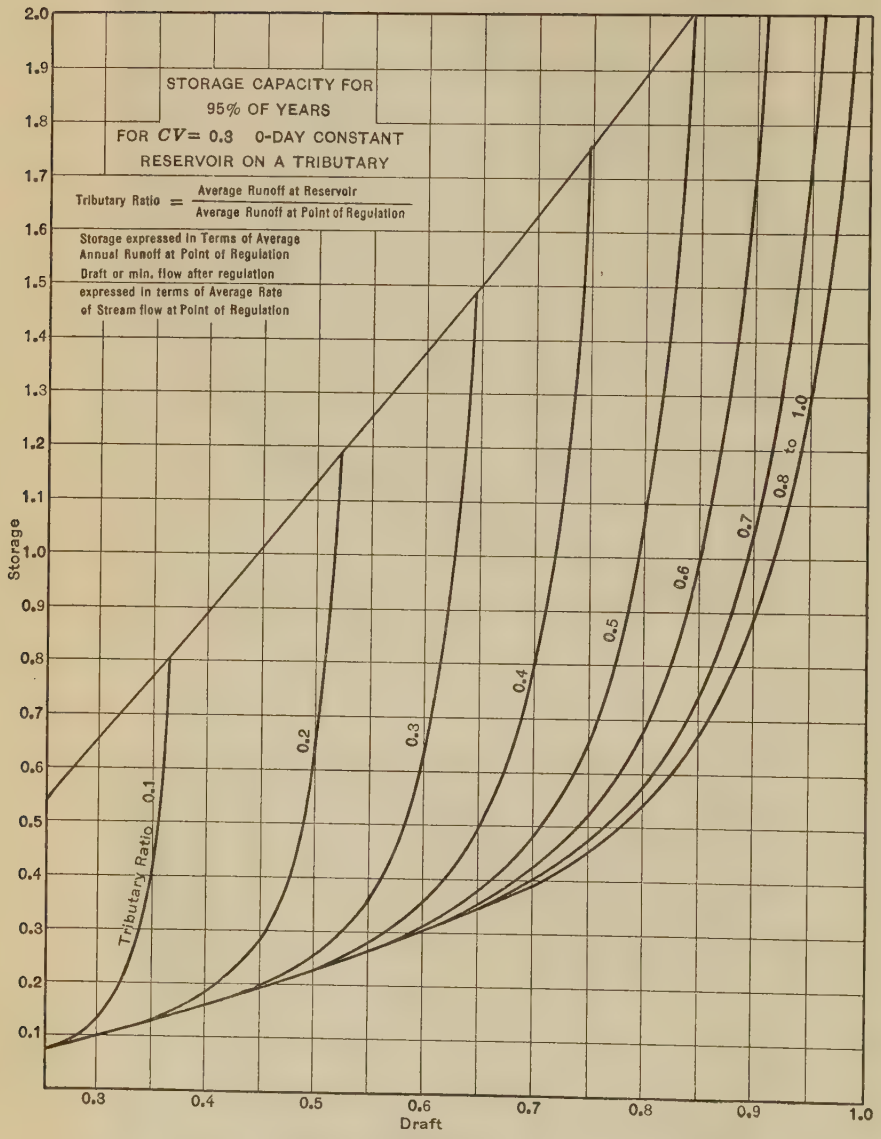
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.3 0-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0

FIG. 31.

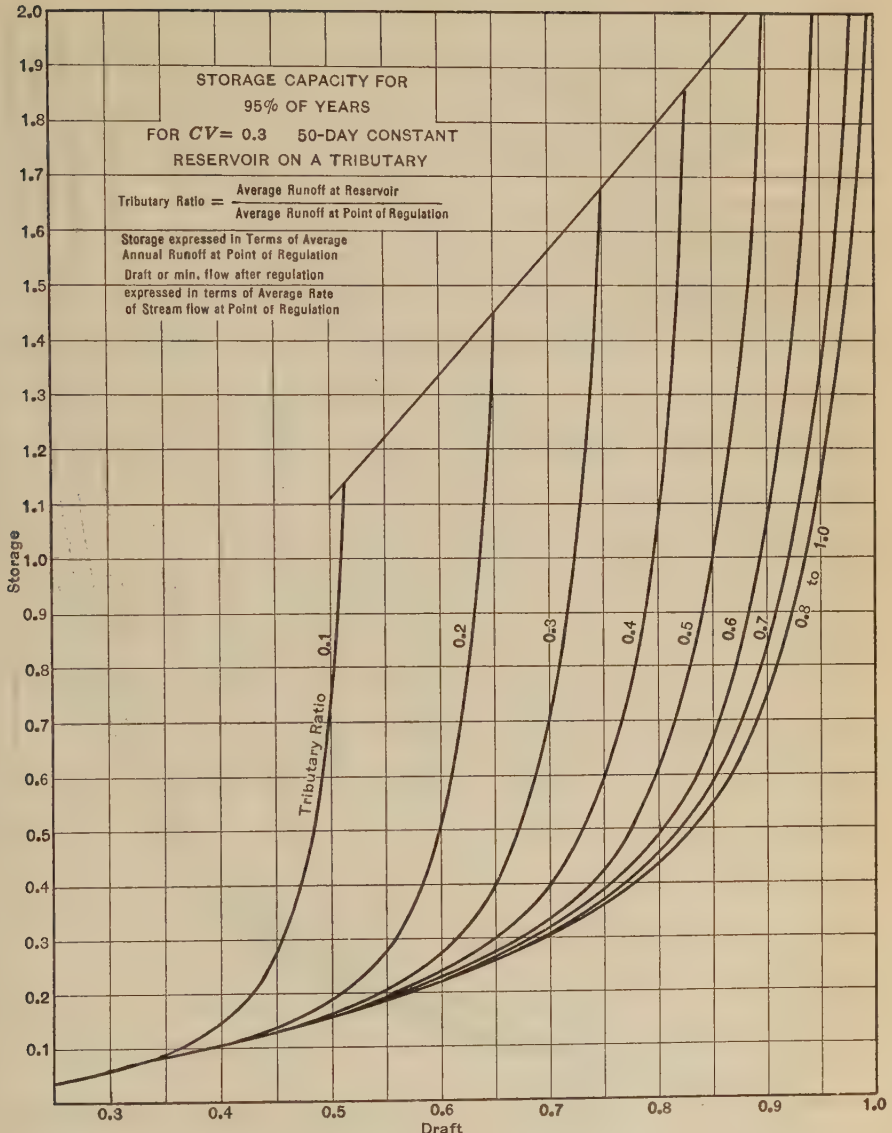
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.3 50-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 32.

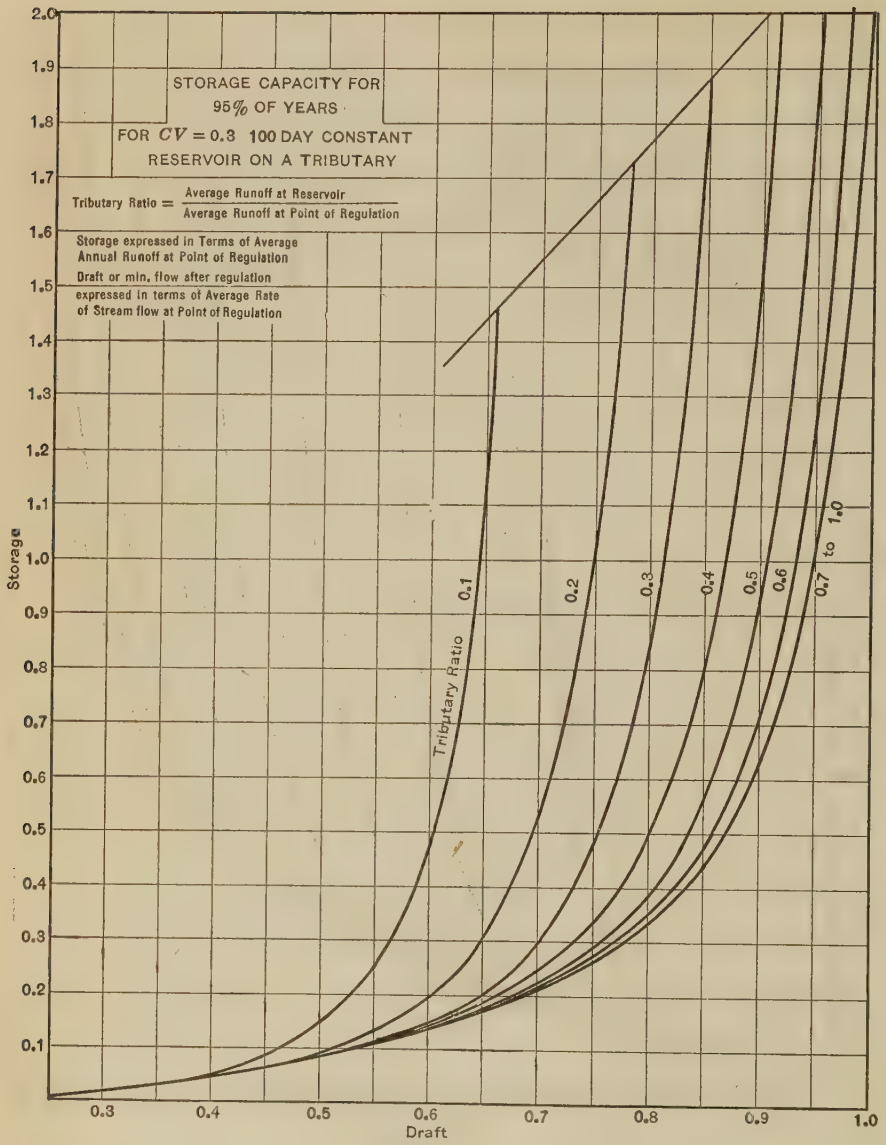
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.3 100 DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7 to 1.0
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 33.

4.—The seasonal storage is affected by the storage constant and the percentage of time that regulation is to be maintained.

Seasonal storage (Figs. 6 and 7) is the total storage required below the limiting draft without annual storage, for all tributary ratios.

5.—The monthly storage is affected by the storage constant, the CV, the tributary ratio, and the percentage of time that regulation is to be maintained.

The monthly storage for any draft equals the seasonal storage (Paragraph 4) minus the annual deficiency (Paragraph 3), both taken for the same draft and the same percentage of time that the draft is to be maintained.

6.—The annual storage is affected by the storage constant, the CV, the tributary ratio, and the percentage of time that regulation is to be maintained.

To compute the annual storage reduce the draft to terms of the maximum possible draft for the given storage constant and CV; and multiply by the annual storage for a 100% tributary ratio as given for the same fractional draft.

7.—The total storage for drafts requiring annual storage equals the monthly storage (Paragraph 5) plus the annual storage (Paragraph 6).

The total storage as computed for 95% regulation is shown in Figs. 25 to 33, inclusive. As there are so many variables—the draft, the tributary ratio, the storage constant, and the coefficient of variation—all affecting the result for each different percentage of time, it has not been found possible to plot the results on one simple diagram. Separate curves, therefore, have been drawn for CV of 0.1, 0.2, and 0.3, and for storage constants of 0, 50, and 100 days. Interpolations, as for example, with the conditions of $CV = 0.16$ and 30 days constant, may be made as follows:

Assume an average annual run-off of 2 000 000 cu. ft.; storage capacity of 1 400 000 cu. ft. (storage = 0.7); run-off tributary to reservoir = 1 000 000 cu. ft. = 0.5 tributary ratio. What draft can be maintained 95% of the time?

Take values for a 0.5 tributary ratio, 0.7 storage, thus:

$CV = 0.10$, 0-day constant; draft = 0.827 (Fig. 25).
$CV = 0.10$ 50- " " ; " = 0.888 (Fig. 26).

For 30-day constant interpolate the draft as 0.864:

$CV = 0.20$, 0-day constant; draft = 0.794 (Fig. 28).
$CV = 0.20$, 50- " " ; " = 0.855 (Fig. 29).

For 30-day constant interpolate the draft as 0.831: For $CV = 0.16$ by interpolation the draft is 0.844; then, $0.844 \times 2\,000\,000 = 1\,688\,000$ cu. ft. annual draft = 3.2 cu. ft. per sec.

Where a number of studies of a stream are to be made, a set of curves applicable to the given storage constant and CV may be computed as in Table 3.

Column (6) of Table 3 may be calculated from the 95% curve (Fig. 13). Thus, for 1.0 draft the storage is $7.4 \times CV$, or $7.4 \times 0.16 = 1.19$. This represents the annual storage in terms of the average available run-off, which is 0.541. The required annual storage is, therefore, $1.19 \times 0.541 = 0.640$. The next draft

represents 0.99 of the available run-off, or 1 — 0.0625 CV, and for this draft Fig. 13 shows a storage of 5.2 CV; then 5.2 × 0.16 = 0.832, which multiplied by the available run-off of 0.541 gives an annual storage of 0.452.

TABLE 3.—COMPUTATION FOR 95% YEAR STORAGE-DRAFT CURVE.

0.1 tributary ratio; $C\,V = 0.16$. 60-day constant; 95% annual run-off (Fig. 12) = 0.768.
Maximum possible draft (Fig. 24) = 0.541.
Limiting draft for seasonal storage = 0.541 × 0.768 = 0.415.
Drafts of less than 0.415 follow the 95% seasonal storage curves (Fig. 6), corrected for 60-day storage constant.

Draft, in terms of maximum possible. (1)	Draft, in terms of average run-off. (2)	Seasonal storage. (3)	Annual deficiency, Column (2) — 0.415. (4)	Monthly storage, Column (3) — Column (4). (5)	Annual storage. (6)	Total storage, Column (5) + Column (6). (7)
1.00	0.541	0.167	0.126	0.041	0.640	0.681
0.99	0.536	0.164	0.121	0.043	0.452	0.495
0.98	0.530	0.160	0.115	0.045	0.355	0.400
0.96	0.519	0.154	0.104	0.050	0.261	0.311
0.94	0.508	0.148	0.093	0.055	0.169	0.224
0.90	0.487	0.136	0.072	0.064	0.095	0.159
0.85	0.460	0.121	0.045	0.076	0.052	0.128
0.80	0.433	0.107	0.018	0.089	0.018	0.107
0.768	0.415	0.098	0.000	0.098	0.000	0.098

TEST OF THE CURVES

This whole discussion rests on several assumptions which are not easily susceptible of proof, while the work of making tests with long-term flow records is very laborious. In making such tests daily flows or average weekly flows should be used, since monthly flows usually will not show the full quantity of water flowing in the main stream at rates exceeding the draft. Some tests have been made using a 20-year flow record of the Hudson River at Mechanicville, plotted in weekly averages. The assumptions as to the tributary ratio are arbitrary and do not refer to any particular reservoir locations. The computed CV is 0.16, and storage constant 60 days, thus necessitating interpolated drafts from the tributary storage curves (Figs. 25 to 33).

1.—The assumed reservoir capacity = 0.46 of the average run-off at the point of use.
The assumed average run-off at the point of use = 1.0.
The assumed average run-off at the reservoir = 0.2 = tributary ratio.
The 95% regulated draft (computed) = 0.65.
Result: The draft can be maintained 18 years, or 90% of the time. There are deficiencies of 0.16 and 0.128 in two of the years and the reservoir is from 95 to 99% empty during several other years.

2.—The assumed reservoir capacity = 0.68.
The assumed tributary ratio = 0.1.
The 95% regulated draft (computed) = 0.54.
Result: This is the maximum possible draft for these extreme conditions. The same draft with the reservoir at the point of use would require only 0.15 storage, which indicates the large quantity of annual storage required because of the location of the reservoir. The draft is actually maintained 19 years, or 95% of time.

3.—The assumed tributary ratio = 0.5.
The assumed draft = 0.87.
The 95% storage required (computed) = 0.60.
Result: The draft is maintained through the 20 years of record, but an actual storage of 0.58 is needed in the twelfth and again in the fifteenth years.

4.—The assumed tributary ratio = 0.1.
The assumed draft = 0.5.
The 95% storage required (computed) = 0.20.
Result: The draft is maintained 70% of the time, with deficiencies of 0.035, 0.029, 0.045, 0.002, 0.012, and 0.088, respectively. One group of three dry years, during which 0.26 storage would have been required, causes the failure to maintain the draft. One of these, the year 1910-11, was extremely dry, the run-off being only 68% of the average.

It seems to the writer that these tests tend to confirm the reasoning on which the tributary storage diagrams are based. They are bound to throw some light on the subject. It is hoped that other engineers who may have occasion to study the question, will subject the data given herewith to like critical tests.

Conclusion

Although duration curves, plotted from flow records are much used in estimating the probable future occurrence and magnitude of stream flows, there are comparatively few American streams which have been satisfactorily gauged for more than ten or fifteen years, and in many instances the engineer is fortunate if even five years of recorded flow are available for his guidance. It is to be regretted that work of this character, of such real utility, does not strike the popular fancy so that adequate appropriations may be made available.

Conclusions based on such short-term records, unless supplemented by additional data, may be considerably in error. The probability curve furnishes a means of extending such records in a most satisfactory manner although it has not received general recognition. Mr. Hazen's "seasonal storage" curves are probability curves, not of flow, but of deficiency based on flow, and they provide simple, rapid means for predicting the amount of storage necessary for release in dry years to maintain a desired use or draft. For all rates of flow and for all percentages of time, only one simple calculation is needed, which may be based on a short-term flow record.

The coefficient of variation, which is the key to the total storage requirements for high rates of draft, likewise may be determined from a short record with small expenditure of labor.

Both these constants for a given stream are based on the usual variations rather than on those extremes which occur at long intervals. It is important to note this fact, for to it is due the accuracy obtainable with short records by the method here advocated. Having computed these two constants all problems of deficiency and storage may be solved by reading from the diagrams here given, with results always consistent and permitting the economic

reservoir capacity or location to be determined with the same ease, for example, as its cost.

The total storage curves presented are not of universal application because they are computed for a coefficient of skew of 0.6, which is suitable only to streams having a *CV* of 0.3, or less. This was shown by H. Alden Foster, Assoc. M. Am. Soc. C. E., in his paper, "Theoretical Frequency Curves and Their Application to Engineering Problems".* A range of *CV* from 0.0 to 0.3 seems, however, to cover most streams of the Eastern United States, and perhaps many other localities.

In his paper, Mr. Foster also shows that while the *CV* may be found with a fair degree of accuracy by means of a short-term flow record, a definitely accurate value of the *CS* can only be ascertained through the use of a record of considerable length. Although further computations of the *CS* for streams in various parts of the world would be of great value, the error in the amount of storage required as based on a somewhat inaccurate *CS* need not be large, and it is possible that only two or three values of *CS* need be selected as a basis for a series of storage-draft curves of practically universal application. The test of annual storage required for the Murray River, having *CV* of 0.52, given by Mr. Foster,† furnishes some ground for this statement.

There is abundant opportunity and room for further research into the apparent vagaries of rainfall, run-off, flood flow, and other problems of hydrology; and certainly there is much to be gained thereby. A comprehensive examination, classification, and study of the most reliable long records of this and other countries, as to their coefficients of variation and skew, their curves of flood flows, etc., while involving considerable labor, would surely prove of immense value in this age when stream control for irrigation, water supply, flood prevention, navigation, and power is of such immediate interest.

The author wishes to thank William Barclay Parsons and John P. Hogan, Members, Am. Soc. C. E., who, to use a popular expression in a literal sense, "engineered" the New York Water Power Investigation; and whose encouragement and interest made this study possible. In the endeavor to find a logical basis for the work and in order to check the resulting curves a large amount of tedious and not always interesting computation was performed by Messrs. H. Alden Foster, H. R. Bouton, J. E. Beswick, and F. G. Bennett, Associate Members, Am. Soc. C. E., and C. A. Wright, Jun. Am. Soc. C. E., whose intelligent and patient co-operation is sincerely appreciated.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

† *Loc. cit.*, p. 169.

DISCUSSION

EDWARD H. SARGENT,* M. AM. SOC. C. E. (by letter).—Mr. Sudler has earned the thanks of the profession for this paper summarizing his studies of the storage required for stream-flow regulation.

For about ten years the writer was in charge of the investigations of the water storage and power possibilities of New York State made by the Conservation Commission. Most of the results of his studies were embodied in the Annual Reports of that Commission for the years 1911 to 1921. Attention is invited particularly to the 1919 Report in which is given a set of duration curves of most of the streams of the State and also curves showing the relation between duration curves plotted from daily and monthly flow values, respectively.

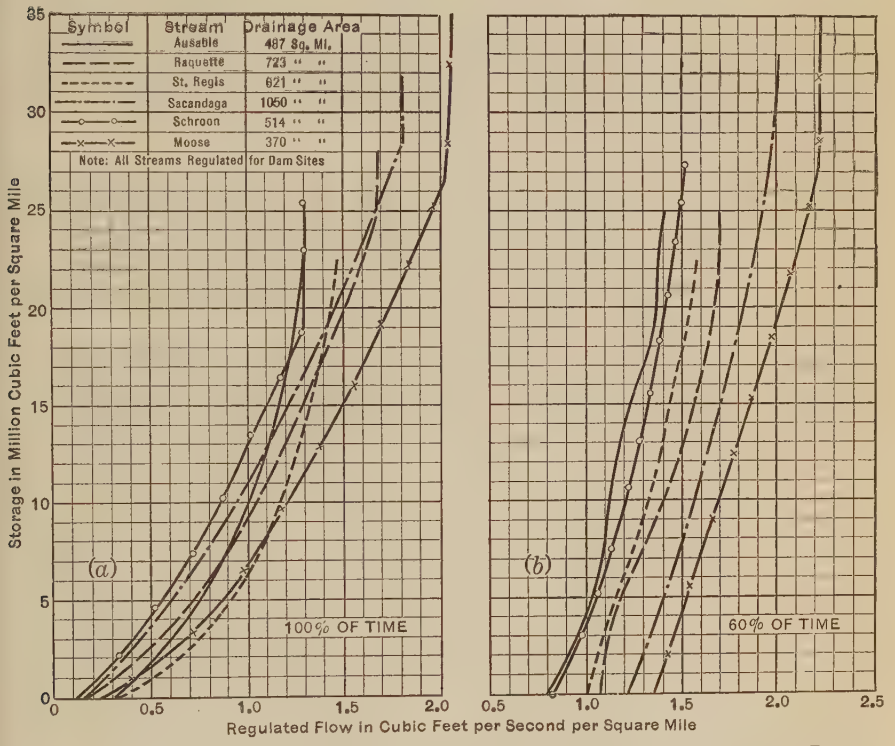

FIG. 34.—CHARACTERISTIC CURVES OF VARIOUS STREAMS, SHOWING REGULATION POSSIBLE WITH VARYING AMOUNTS OF STORAGE.

Herewith are submitted two diagrams (Fig. 34 (*a*) and Fig. 34 (*b*)) relating to stream flow regulation, typical of many similar diagrams, which were not published. Fig. 34 (*a*) shows the regulation that could have been maintained 100% of the time, that is, continuously, on various streams with given amounts of storage. These curves were plotted from data obtained from mass and

* Chf. Engr., Hudson River Regulating Dist., Albany, N. Y.

duration curve studies of the streams in question. Fig. 34 (*b*) is similar to Fig. 34 (*a*), except that it shows the flows that could have been maintained 60% of the time for given amounts of storage.

It was the writer's intention to prepare and publish in the reports of the Conservation Commission similar curves for all streams in the State of which there were reliable stream-gauging records; but the decision of the Legislature to cease making appropriations for water power investigations prevented this. It was believed that these curves, used in conjunction with comparative hydrological and topographical data, would have afforded a quick means of making storage and regulation studies.

The writer's thoughts as to stream regulation for the combined purposes of power development and flood control were given in his discussion* of the paper by J. C. Stevens, M. Am. Soc. C. E., entitled "Stream Regulation with Reference to Irrigation and Power",† as were also his studies of storage reservoir operation. Subsequent studies along this line have led to the belief that careful examination of the hydrology of a water-shed will make it possible in the actual operation of a reservoir to get from 85 to 90% of the so-called "ideal" or theoretical regulation.

Allen Hazen,‡ M. Am. Soc. C. E. (by letter).—This paper contains a contribution of real importance to the methods of making storage calculations. The use of a mass curve, in which is shown only the accumulated surplus or deficiency as compared with the mean, permits a more convenient representation and accurate study of the data. This is particularly true for high rates of draft.

The author shows the method applied to annual quantities, but it is equally applicable to monthly quantities and can be used for the whole storage calculation for the higher rates of draft. The calculations may be prepared for plotting on calculating machines rapidly and accurately and with full checking. After having tried this method on an example the writer wonders that it was not done long ago.

It may be recalled that in the writer's paper entitled "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply,"§ published in 1914 and subsequently herein referred to as the paper of 1914, storage was divided into two parts, namely, seasonal storage, or the storage required to hold the water of the wet part of any one year and make it available in dry months next following, and, annual storage, or the storage of water held more than one season and finally drawn in a year when the rate of draft exceeds the total mean flow for that year.

For storages up to the mean flow of a dry year, seasonal storage is all that need be considered. For higher rates of draft an addition for annual storage must be made. For rates of draft closely approaching 100%, and especially in the West where flows are less regular, the annual storage becomes the largest element in the calculation.

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* *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 971.
† *Loc. cit.*, p. 958.
‡ Cons. Engr. (Hazen & Whipple), New York, N. Y.
§ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

The author checks both the methods of calculation and the amounts of seasonal storage of the paper of 1914 in a way that is highly gratifying. This part of the storage, especially for Eastern streams, frequently represents the whole range of use and interest. With annual storage, however, and in the methods of combination of seasonal and annual storage, divergences are found which call for study.

In the 1914 paper the two kinds of storage were simply added. The question was then raised as to whether or not this procedure was accurate.* Subsequent study showed that it could be improved. The highest annual storages and the highest seasonal storages did not occur in the same years. If the seasonal storages were arranged in the order of magnitude and the annual storages were placed opposite the various years as they occurred, it was found that the annual storages were not all grouped at the upper end, but that some were scattered all through the series.

The simple addition indicated too much storage for the highest rates of draft. Trials were made of adding the annual storage for a 95% dry year to the seasonal storage for a 70% dry year to get approximately the total storage for a 95% year; and also the other way about, of adding the seasonal storage for a 95% dry year to the annual storage for the 85% dry year to produce the same result. Both gave results that came somewhat closer to the facts.

The test of the accuracy of the procedure has been that the sum of storages computed for the higher rates of draft must be equal to the sum of the corresponding actual storages for a number of streams with the longest records. The comparison of one or two streams is not enough. The longest records at hand are not long enough to smooth out the irregularities; the considerable variations between results obtained from different streams are considered to be matters of chance and the average of all is taken as the criterion.

The next method tried was called the square root method. If the two kinds of storage followed exactly the normal law of error, it would be possible to combine them into a single series by a mathematical process, namely, by adding the median values and finding the standard variation for the combined series as the square root of the sums of the squares of the standard variations of the two separate series.

As the curves indicating storage are not normal, but are more or less skewed, the calculation can only be applied to parts of those series so short that the deviations of those parts from the normal law of error are not important. This can be done by taking the ends of the curves representing the dryest years of a series, where they are approximately straight, and continuing them as straight lines across the diagram. Figures taken from these lines may be combined as previously. It was found that this method gave results agreeing for very dry years with the actual required storages computed from the long-term records of a few streams, but for less dry years there were divergences that could not be overlooked.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), pp. 1602–1603.

The reason for these divergences turned out to be that in the series representing annual storage many of the terms, frequently more than half of them, as shown by the extended straight line, were negative. Actually, the negative terms do not exist. The calculation had been made as if they did exist and the effect was that these theoretical negative values pulled down the results and the calculated figures were too low except at the very end for the dryest years, when there are never negative values.

Afterward, an effort was made to adjust these differences by using the foregoing method to find the value for a 99% dry year and another method to fix a second point. The second point was obtained by plotting the sum of the means of the two series on the 56% line, the latter being based on an actual count of terms in comparable data for Eastern streams. These two points were then connected by a straight line on arithmetic probability paper and values for the 95% dry year, and other desired intermediate points, taken from it.

In applying this method a rather long series of simple calculations and drawings were made once for all, and the tables and curves prepared, which have since been checked with actual records of as many streams as have been found available. This method, although arbitrary, is the most satisfactory that the writer has found. Tables and curves prepared in this manner have been published.*

The writer feels that this method has been tested by practical experience to the point where, within reasonable certainty, it is in accordance with the facts and is a safe working method; but a better theoretical method of combination is to be desired.

The author's estimates for required storage for the higher rates of draft where annual storage is involved are much lower than those reached by the writer in 1914, and in his subsequent work. The difference seems to be one of definition.

The original method was to assume an indefinitely large reservoir full at the beginning of the record period; to compute the maximum depletions occurring in successive annual periods; and to make a series of these depletion values. The amount of storage required to maintain service in a 95% dry year was taken directly from the curve of these results. In other words, with an assumed constant draft and unlimited storage, the storage shown by the calculation is that which would be exceeded during 5 years in 100, or in whatever other ratio was desired.

The author has followed another assumption, namely, that the reservoir was of limited size (the size to be found by the subsequent procedure) and that, when it was once emptied (by failure to maintain full service), the record up to that point was closed and a new calculation was started with the reservoir empty and carried forward in the following years.

L. Standish Hall, Assoc. M. Am. Soc. C. E., made† assumptions in regard to this on a still different basis, which also produced results smaller than

* American Civil Engineers' Pocketbook, 4th Edition, 1920, and American Water Works Association Manual of Practice, 1925.

† *Transactions*, Am. Soc. C E., Vol. LXXXIV (1921), p. 249.

those reached by the writer. As a basis of statement and calculation, any assumption may be made, and it is a fair field in which each has his choice. As a matter of practical application and procedure, in order that the results may be useful and safe, it is necessary that the assumptions should be reasonably in accordance with the conditions of actual use of the structures that will be effected by the calculations.

Using the writer's method and applying it to a 95% dry year, which seems to be the best ordinary basis of rating water supplies in the United States, there is a probability, as stated in the Water Works Manual, that the full supply can be maintained for 95 out of each 100 years and that in the other 5 years, there will be shortages with deficiencies ranging from 1 to 10% (to somewhat more than 10% at long intervals) and averaging about 6% for all the years of shortage, these being 5% of the whole number.

With the assumptions made by the author it is clear that smaller reservoirs would result and the deficiencies for the years of shortage would be more than 6% and might occasionally be very much more. The author may have had power-houses in mind in making his assumption, but the writer confines his remarks to calculations for public water supplies. It may be that this difference in point of view goes to the heart of the matter.

In the writer's conception of the problem, there is no thought of stopping the draft for any interval when the reservoir is empty, or of limiting it to the natural flow of the stream for that period, or to any other greatly reduced amount. Instead it is his idea that as storage becomes depleted, consumption will be curtailed by measures known to water-works men, such curtailment not exceeding about 10% of the normal output, and that the storage provided must be sufficient to maintain such slightly reduced service throughout the whole dry period and until larger flows are again available.

The author has kindly furnished in his paper the necessary data for a comparison. Referring to total required storages on the Croton River, and passing the higher rates of draft which are considered impracticable or inadvisable in the water-works business, it is stated that, for a draft of 0.9 of the mean flow, the computed storage by the author's method is 0.69 of the mean flow for 1 year. From the actual record for 47 years he finds that this storage would maintain the desired rate of draft for 43 years, or 90.4% of all, with deficiencies of 2, 5, 22, and 11% in the 4 years of shortage; that is, instead of actually maintaining the supply for all but 5% of the whole number of years, the calculated amount of storage falls short in 9.6% of the years, or twice as often as it should. Moreover, by the author's calculation, in 1 year there would be a shortage of 22% in available supply which is an intolerable assumption in public water supply.

For comparison with this, following the method of the American Civil Engineers' Pocketbook or the American Water Works Association's Manual of Practice, the required storage estimated by the writer for that draft is 0.93% of 1 year's mean flow. The storage over that computed by the author, 0.93 instead of 0.69, an excess of 0.24, is sufficient to absorb the entire deficiency found by the author for the one year of greatest shortage.

The matter may be shown graphically. Various storage lines have been drawn on Fig. 35. These calculations all relate to one assumed case, but the case is a representative one. Line 1 shows the seasonal storage and it is in regard to this line that there is complete agreement between the author and the writer. Line 2 shows the annual storage as computed by the writer, that is, the storage required to carry the dry years. This is over and above and does not include seasonal storage. It is separately computed by a procedure described in the paper of 1914. Line 3 is that found by the addition of numbers in Lines 1 and 2 and represents total storage as used by the writer in 1914.

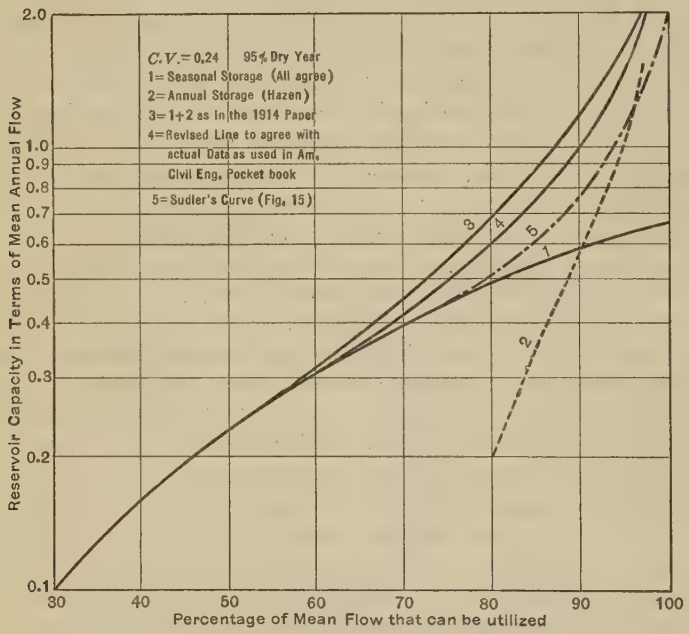

FIG. 35.—STORAGE DIAGRAM.

Line 4 is plotted from the figures in the American Civil Engineers' Pocketbook, and results from the application of the procedure that has been briefly outlined. It will be seen that the rating of a source by using this line will be greater for all the higher rates of draft and sometimes will be as much as 3% greater than that found by using the procedure of 1914. Line 5 is plotted from the values given by the author.

The suggestion is made to water-works men that the values for storage given in the American Civil Engineers' Pocketbook will probably be more comfortable to use in the long run.

The study of storage on a tributary is interesting. The writer has an arbitrary rule which seems to be in substantial agreement with the diagrams presented by the author. This is to the effect that storage on a tributary on Eastern streams up to about 0.9 of the mean flow is substantially as useful

as a like amount of storage in a reservoir controlling the entire catchment. Additional storage is less useful and even a very large reservoir on a tributary is not useful beyond the equivalent of one year's mean flow to it.

ROGER W. ARMSTRONG,* M. AM. SOC. C. E.—The author's method of computing the coefficient of variation is novel and interesting. The utilization of "storage" years, which are intervals between times of like conditions in the run-off cycle and may be of varying length, undoubtedly produces a more accurate value of the coefficient than may be obtained by the use of calendar years. This draws attention to the fact that in arithmetical computations of deficiencies of flow some inaccuracies in the values obtained may result from using calendar years rather than "storage" years. Whether such errors would ever be of sufficient importance to justify the greater amount of labor required for computing with "storage" years, is a question. Of course, in determining deficiencies in flow by graphic methods with a mass curve, the "storage" years are used automatically.

The values of the coefficient of variation of the Croton River (page 644), are significant. They seem to indicate that a reliable value of the coefficient cannot be obtained from a record of fewer than about twenty years. It would be of interest to have the author's opinion as to the shortest length of record necessary for the determination of a satisfactory coefficient of variation.

H. ALDEN FOSTER,† ASSOC. M. AM. SOC. C. E. (by letter).—The writer was rather closely connected with many of the original studies on which this paper is based, and therefore has examined it with a great deal of interest. In reviewing the analysis and methods given by the author, after an interval of several years, the writer is again convinced that they are logically developed from the general assumptions on which they are based. These assumptions may be summarized as follows:

1.—The duration curve of daily flow for the 95% dry year (or any other dry year) is directly proportional to the corresponding curve for the average year, the proportional factor being the ratio of the total flow in the dry year to the total flow in the average year.

2.—The mass curve of daily flow for the dry year is directly proportional to the corresponding curve for the average year, with the same proportional ratio as before.

3.—The year requiring the greatest seasonal storage also produces the lowest total run-off.

4.—The maximum annual storage is required in the same year in which the seasonal storage is a maximum.

It would probably be impossible to prove definitely that any one of these assumptions is true in all cases. On the other hand they undoubtedly are descriptive of general conditions in Nature. A theory based on these assumptions must be considered, therefore, as reliable in a qualitative or relative

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† With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

manner, but not as giving absolutely precise results in any given case. The writer believes that the tests given in the paper clearly illustrate this point.

Moreover, as the author indicates, a stream-flow record covering only a few years is certainly not long enough to show, by itself, what the true storage requirements for that particular river would be over any considerable period of years. For such a purpose, the results published by Mr. Hazen,* and used by Mr. Sudler in preparing his seasonal storage diagrams, are of great value. The method by which the author combines seasonal with annual storage is logically correct, and an improvement over previous methods.

The only point of view from which objection might be raised to the author's analysis is in connection with the development of the annual storage curves. The determination of the required annual storage is undoubtedly a very difficult problem, and one for which it may be impossible to give a universal solution. It can only be determined by a study of accumulated long-term records, or by the construction of an artificial record, such as that described by Mr. Sudler. Whether one method of deriving annual storage curves is more reliable than another would be largely a matter of opinion. The writer believes that the annual storage curves presented in the paper are entirely consistent, and as reliable as any that can be developed from the present available data.

These annual storage curves were developed from the theoretical duration curve of annual run-off described by the writer.† It was brought out in the discussion of that paper that there are other types of duration curves which are more applicable to certain streams than the curve originally presented. If a different duration curve had been used by the author, his annual storage curves would probably have been slightly different. It should be noted, however, that two duration curves having the same coefficient of variation will not differ greatly for values less than the mean run-off, up to 80 or 90% of the time. As annual storage usually is required for a group of consecutive dry years, the maximum annual storage is as likely to be required for a series of moderately dry years as for a short group including the extremely dry years. Consequently, a change in the duration curve of annual run-off which only affects the lower end of the curve but does not materially change the more numerous points closer to the average run-off, will not have any great effect on the annual storage requirements.

This same line of reasoning should apply to small changes in the coefficient of skew, for a change in this coefficient has a much greater effect on the extreme ends of the duration curve than on the intermediate portion. Moreover, an increase in the *CS*, while it increases the number of years with deficient flow, at the same time increases the flow in the dryest years. It may be stated, therefore, that the curves prepared by the author, which were computed for a coefficient of skew of 0.6, should be applicable to streams the *CS* of which is between 0.3 and 0.9, or possibly for an even greater range.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

It is, therefore, the writer's opinion that Mr. Sudler's curves may be used for streams having a considerable range in run-off characteristics. Even for streams that are clearly outside the range of these curves the diagrams might be used for comparative purposes, where precise quantitative results are not essential.

In any case, one of the important uses of the diagrams should be for the purpose of studying the relative effects of different reservoir and power-house sites on a particular stream. The results to be obtained for this purpose may be considered as entirely consistent, and as having a large degree of precision when the characteristics of the stream are within the limits specified for the diagrams.

JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).—The methods of studying storage possibilities developed by Mr. Sudler will doubtless find considerable application where regulation for maximum dependable flow is required and where a large number of storage possibilities must be studied. In such cases the method should greatly shorten the rather laborious computations that are usually made.

However, after the reservoir and its size have once been determined, it will always be advisable, the writer believes, to follow through, by the old-fashioned methods of computation, the operation of the reservoir month by month (and even day by day during critical periods) for the period of record. Such computations show the water added to the reservoir, the water released to maintain the required flow at the given point, and the resulting elevation and contents of the reservoir. They have the advantage of showing what the reservoir would have accomplished had it been in operation during the years of record.

Whereas engineering studies and investigations are conducted to determine the truth, the resulting report is usually presented with the object of either getting some one to spend money or to prevent him from spending it. The old-fashioned methods of computation will always have, the writer believes, merit for inclusion in such reports, as they have the advantage of being easily understood by all engineers and most power company executives.

The method of building up an artificial long-term record as used by Mr. Sudler by chance drawings from the years of actual record, is certainly open to some objections. The variations in rainfall and run-off from year to year on a given water-shed are largely unexplained, but this does not mean that the sequence of wet and dry years is wholly a matter of chance Any method of extending a run-off record is open to objections. Although such records are often very short, it is usual to have rainfall and meteorological data available for a much longer term of years. By the proper use of such data, considered in connection with the existing run-off records, the writer believes that it is usually possible to build up reasonably accurate records of synthetic run-off, extending the actual records backward for many years.

Short as most run-off records are, many of them are long enough to cover marked changes in the physical character of the water-sheds. Water-sheds

* Hydr. Engr., U. G. I. Contr. Co., Philadelphia, Pa.

which, when records were started, were well forested, now have had these forests replaced by brush or farms, towns, and cities. In other cases, lakes and swamps have been drained and, in others, reservoirs have been added. Such changes, however, probably affect the distribution of the run-off throughout the water-year more than they do the total amount of run-off.

The paper deals with the problem of maintaining the maximum low-water flow either at the reservoir outlet or at some point down stream where the water-shed area is much larger. With the growth of power companies and their extensive interconnection by means of long transmission lines the problem of storage from the standpoint of the engineer engaged in hydro-electric work is rapidly becoming much more complicated. The usual problem is no longer to maintain a maximum dependable flow at some given point on a stream, but rather to release the water in the reservoir so that it will produce the maximum in firm power and primary energy in a power system which may embrace many hundred square miles of territory and extend over several States.

Such a study may embrace not only the reservoir and the hydro-electric plants on the stream in question, but also a considerable number of interconnected hydro-electric plants on other widely scattered streams, as well as existing storage reservoirs and steam plants. Such a method of use when practicable will show a much greater income creditable to the storage reservoir than by using it to produce the maximum dependable flow at some particular point on the stream. This method of use also results in over-storage; that is, the economic size of reservoir will be larger than when regulation is for maximum dependable flow. In fact, the water in the reservoir is considered as so much potential energy to be used where it will do the most good in increasing the firm power in the system. The whole stream on which the reservoir is located becomes a storage reservoir of energy, as it were. For this kind of regulation, which is becoming increasingly important, an ideal stream would be one possessing one or more economical sites for storage with a maximum amount of head capable of economic development in relatively high-head plants below the storage site and with all sites under one ownership or under sympathetic ownership.

A relatively simple case will be used to demonstrate this method of regulation. In Fig. 36, River *X* has a large storage reservoir; also located on it are Plants A, B, and C, which are high-head plants with relatively short penstocks, (that is, low cost per kilowatt of installed capacity). The storage reservoir is larger than would be required to maintain maximum dependable flow at, say, Plant B; and Plants A, B, and C have installations much greater than would be justifiable if River *X* were the company's only source of power.

River *Y* is a much larger stream the water-shed of which may be at a long distance from River *X*. This river is uncontrolled, has a relatively flat slope, and is subject to wide fluctuations in flow. Plants D, E, and F are controlled by the same company which controls Plants A, B, and C, on River *X*. The outputs of all the plants feed into the same system.

During periods of minimum flow on River *Y* a maximum quantity of water is released from the storage reservoir on River *X*, and Plants A, B, and C are working at full capacity during the peak hours. Perhaps, during 50% of the time over a term of years, some water will be released from the storage reservoir on River *X*, and during the remainder of the time there is very little energy produced at Plants A, B, and C on River *X*.

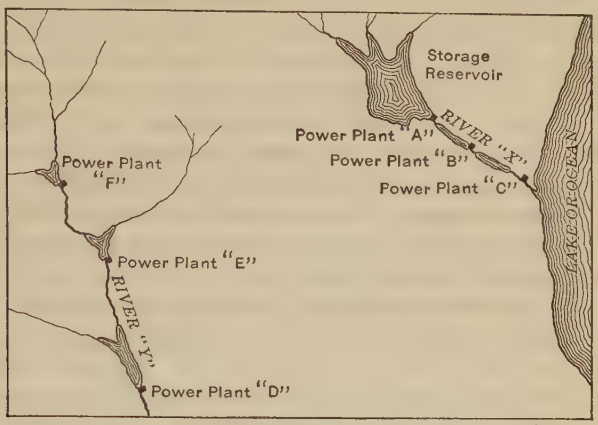

FIG. 36.

On River *Y* Plants D, E, and F have their low-water output augmented by Plants A, B, and C, on River *X*. During the remainder of the time the plants on River *Y* carry the load (except for short-time peaks which may be carried by Plants A, B, and C).

In some cases, plants of the character of A, B, and C which operate only 10% of the time are found to be economically justifiable. Of course, every practical case has many more complications, but these suppositions serve to illustrate the principle of the operation of storage reservoirs for the maximum production of primary energy in a system.

Steam plants often enter into storage studies to a surprising extent. In one case the writer found that it was economical to draw down a certain reservoir every two or three years and on those occasions put into service for a brief period of time an antiquated steam plant which would otherwise be junked. Possibly, there are many cases where this procedure might pay, as it makes the reservoir water do more work and leaves less dead storage in the reservoir. Furthermore, every large power company has several old steam plants about ready to be junked, but which could be kept in condition ready to operate at, say, a month's notice without too great an annual stand-by cost.

L. STANDISH HALL,* ASSOC. M. AM. SOC. C. E. (by letter).—The author is to be congratulated for his timely paper on this important subject. In a recent discussion† of the paper by R. D. Goodrich, M. Am. Soc. C. E., entitled "Straight Line Plotting of Skew Frequency Data", the writer called attention

* Chf. Hydrographer, East Bay Municipal Utility Dist., Oakland, Calif.
† See p. 57.

to the fact that the recent developments in the treatment of skew frequency curves would permit of the improvement in the results obtained in applying probability methods to the study of storage requirements.

One of the principal difficulties encountered in determining storage requirements is the proper combination of seasonal and annual storage. If the draft to be used is less than the minimum annual flow of the stream, the study is greatly simplified, as in this case only seasonal storage need be considered. For the purpose of determining the amount of such storage, the curves developed by Allen Hazen, M. Am. Soc. C. E., appear to be satisfactory.*

The new term introduced in the paper, "monthly storage," appears to be unnecessary. In Fig. 1, showing a typical mass curve, the author has considered the years to be divided by the peaks. Using this division of the year, the monthly storage term becomes necessary in computing the increment to be added to the annual storage. However, such a division of the year (called by the author the "storage year") does not coincide with the normal water-year. The water-year begins either in the fall, or else coincides with the calendar year, and includes all the run-off occurring in one annual cycle. An annual cycle should logically include the run-off occurring between periods of low summer flow.

Considering the mass curve in Fig. 1, the storage is drawn down during the first year by the seasonal storage indicated by the ordinate, *a*. In the second year the reservoir did not refill, and at the end of that year the depletion of the reservoir at *b* is the total ordinate between the draft line and the mass curve. The amount of annual storage is this ordinate minus *a*. Similarly, in the third year, the annual storage is the ordinate *c* plus *e* minus *a*. In other words, the amount of seasonal storage to be considered in any period of depletion of a reservoir, extending over two years or more, is the amount occurring in the first year. It is evident that the seasonal storage, *a*, is independent of the succeeding years and must be caused by a year having a run-off greater than the rate of draft. As any such year might begin a period of drought, a logical method of allowing for seasonal storage would be to add to the annual storages, the average seasonal storage required in years having flows above the rate of yearly draft. It is possible that further study of the probabilities involved in the combination of the two quantities may disclose some better way of combining annual and seasonal storage.

One weakness of the author's method is that, with high rates of draft and depletion of the reservoir extending over many years, the maximum depletion may not be coincident with the year of minimum run-off. This is the assumption that the author makes in his method. In Table 1, the maximum depletion occurs in the sixth and the tenth years, the sixth year being the year of minimum run-off. In this table, a very slight change in the run-off percentages would cause the maximum depletion to fall in the tenth year alone, and this year is only slightly less than the rate of draft. Such a condition frequently occurs with the higher rates of draft.

* "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

H. Alden Foster, Assoc. M. Am. Soc. C. E., has shown* that with a frequency distribution of a given type under the Pearsonian classification (that is, Type I, Type III, etc.) having a certain *C S*, the variations from the mean are proportional to the *C V*. The author has evidently selected the Type III curve as representing the frequency distribution to be applied to stream-flow records, since he states that the *C V* must be less than one-half the *C S*. The writer has shown† that the majority of streams having long records fall in the Type I classification. However, Mr. Foster believes that with longer records more streams would fall in the Type I class.‡

The writer recently combined the records of eighteen streams having a *C S* of approximately 0.50, by reducing the records to a *C V* of 0.10. The resulting series contained 608 terms. It was found that the frequency curve formed by this series lay between the Type III curve and the modified Type I curve, the curve still belonging in the Type I class. The Type I curves have a greater percentage of terms below the mean than the Type III curves, although the minimum values are lower in the case of the Type III curves. It may be that, for the purpose of calculating storage requirements, it is immaterial which type of curve is used.

For a complete analysis of the subject, after the selection of the proper type or types of curve to be used, computations of annual storage should be made for several values of *C S* covering the range of this coefficient ordinarily encountered in stream-flow records. If the annual storage is computed for various values of *C S* and a uniform *C V*, the storage for a given *C V* and *C S* can be easily determined. The author's calculations have been made for a *C S* of 0.60 and will apply to streams falling in this classification. His statement that the curves will apply for *C V*'s of 0.10 to 0.30, is not necessarily true, since a stream having a *C V* of 0.10 might have a *C S* of only 0.20 and still fall in the Type III class.

The author's method of preparing an artificial record is open to criticism. In making the drawings, each card should be returned to the pack before a subsequent drawing is made. By the author's method a correlation exists between each drawing; in other words, after the first drawing the "universe" from which drawings are made is altered.§ The mean for each 50-year period is also equal to the mean of the "universe", whereas under the normal method of drawing this would not be likely.

In plotting the annual storage, the author has used the depletions of the reservoir at the end of each year. The writer feels that this method gives undue weight to periods of maximum depletion of the reservoir, and that only the maximum depletions of the reservoir during each period of draw-down and refilling should be considered. The writer has discussed this subject rather completely in a previous paper‖ and will not repeat it here. He has altered his views in some respects during the meantime, but in the main, the

* "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

† *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 187.

‡ *Loc. cit.*, p. 201.

§ "Introduction to the Theory of Statistics," by G. Udny Yule, pp. 335–336.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 249–255.

procedure there outlined is still applicable. If an artificial record is prepared in the proper manner, as explained, and depletions in storage are computed, it will be found that, for the higher rates of draft, the storage required in the 95% year will occur only during cycles of drought at periods less frequent than once in 100 years.

Summing up the writer's views, there are two points which require especial consideration: First, as to the proper type of curve to apply to stream-flow records—it may be that further study of long records may show that stream flow can be represented by one type of curve, and that the apparent variations from this type are of an accidental nature due to the shortness of the records.* This would necessitate a correlation between the *C V* and *C S*. On the other hand, it may be found that no material difference in storage requirements would be occasioned by the use of either the Type I or the Type III curve; second, further investigation is needed as to the best method of combining annual and seasonal storage.

Edward M. Craig, Jr.,† Assoc. M. Am. Soc. C. E. (by letter).—The mean law of error or the probability of variation from the true, or normal, or average, is not applicable to the probable yield of water-sheds as it is, for example, to the probable error in surveying measurements in at least one important respect. The point of difference is that the number of dry years is usually greater than the number of wet years as measured by variation from the average; and the yield of the wettest year is well known to exceed the average yield by more than the amount the dryest year is deficient.

Sir Alexander Binnie, the English authority, states that from a study of 7 000 rainfall records over the world the annual rainfall is above the mean in 45.8% of the years and below the mean in 54.2% of the years, the average of the wet years being 119% and of the dry years 83 per cent. The yield of the Croton water-shed in 58 years has been above the mean in 44.8% of the years and below the mean in 55.2% of the years. During 23 years of records for the Catskill and Schoharie water-sheds, the same results are true, 52.2% of the years being wetter and 47.8% dryer than the average. The greatest yearly yield of the combined Catskill and Schoharie Systems has been 150%, while the lowest yield has been 80% of the average. For the Croton, the maximum and minimum yearly yield has been 152% and 52%, respectively, of the average yield.

Where the data are sufficient, it is more accurate to use two coefficients of variation, one for the wet years and one for the dry. The coefficient for the wet years will be larger and will indicate a greater variation from the mean in a smaller number of years, while that for the dry years will be smaller and will give less variation from the mean in a greater number of years, which is in accordance with the facts.

It is more doubtful whether the period of maximum droughts can be provided for strictly according to the probability curve. There is a tendency for dry years to succeed dry years and the critical drought that must be pro-

* See pp. 50–53.

† Asst. Engr., Bureau of Water Supply, Dept. of Water Supply, Gas and Electricity, New York, N. Y.

vided against is usually found to be two, three, or four years, or even longer periods. When it is possible it is much safer to consider the droughts that have actually occurred and that may apply to a given region. Burgess* has tabulated the periods of lowest yield that have been known to occur on streams in Northeastern United States. This has advantages on the side of safety over merely using the mass curve for a single stream. It has the disadvantage that it may not take into sufficient account peculiar and local climatological conditions.

Unless the storage ratio is very high, the application of the theory of probabilities emphasizes the lack of absolute security of the so-called safe yield, when it approaches the average yield. There is a big difference between the 100% safe yield and the 95% safe yield. This emphasizes the usefulness and even the necessity of an auxiliary source of water supply, such as wells, the temporary filtration of an impure source, or a connection with the supply of another city, for which it is apparent the municipality could afford to pay ten or twenty times as much as for the water used regularly when it is necessary to make good a deficiency.

W. W. Brush,† M. Am. Soc. C. E.—The speaker is not in a position to discuss critically the development of the curves which Mr. Sudler has shown, but wishes rather to direct attention to a closely related subject—the question as to what represents a proper amount of storage to obtain a certain yield from any stream or any group of streams.

In connection with the water supply of New York City an important problem is the determination of the safe draft each year, and in the different seasons of the year, from a water-shed with a given storage development. The safe draft should not be confused with the safe yield of a water-shed, which is considered to be a fixed amount whereas the safe draft varies with the volume of stored water and the season of the year.

The use of the term "safe yield" of a stream is regrettable. Usually, in discussing the question of the available water supply from any given development, it is said that the "safe yield" is a certain amount. The layman interprets the term "safe" in the sense that he ordinarily hears it applied and believes that if the figure given is not exceeded, there is no question as to the supply being sufficient at all times. This is not so. Since Alfred D. Flinn, M. Am. Soc. C. E., calculated the so-called "safe yield" of 336 000 000 gal. per day with the present storage in the Croton water-shed that figure has been generally adopted as the dependable yield.

In the Catskill System with the present storage development the "safe yield" has been set at between 585 000 000 and 600 000 000 gal. per day. The citizen of New York who reads such a statement believes that he can depend on 336 000 000 gal. daily from the Croton, and on, say, 600 000 000 gal. per day from the Catskill System; and that there need be no worry about drawing that quantity of water from those two sources.

* *Engineering News-Record*, 1922, Vol. 89, p. 970.

† Chf. Engr., Bureau of Water Supply, Dept. of Water Supply, Gas, and Electricity, New York, N. Y.

The engineer responsible for the daily supply of water to New York City would be open to severe criticism if he should proceed on the theory that the so-called "safe yield" could be drawn. As is well known to engineers, the "safe yield" is the quantity of water that could be drawn daily from any source provided that no more severe drought were experienced than the one assumed; and provided that, on the very day the water stored in the reservoirs was entirely exhausted, the necessary rain were to fall to raise the stream flow above the rate of draft. Such a depletion of the stored water in any water supply system would be extremely hazardous.

Both Allen Hazen,* M. Am. Soc. C. E., and Mr. Sudler have pointed out that it is much more accurate to give a percentage of the years in which a certain yield can be depended on and not to call any yield the "safe yield."

The speaker is unable to offer a satisfactory substitute for the expression, "safe yield". He believes engineers should give up the use of that term because it is misleading; and should endeavor to express more accurately to the public what is meant when it is said that a certain yield can be obtained from a certain water-shed with a certain amount of storage development.

The curves that the New York Water Department use in operating are not along the lines developed by Mr. Hazen or Mr. Sudler. Mr. Hazen's method has been used in computations to determine about what percentage of years the so-called "safe yield" could be depended on. The Department's limiting curve for draft has been obtained by a comparison between the rainfall and flow records of the Croton and Catskill water-sheds and the application of the Croton long-time record (nearly 60 years) to the Catskill streams, where a record of 23 years is available.

The storage required at the beginning of each month in the year to maintain a certain rate of draft is determined. The resultant curve for the entire year is plotted. If the water in storage exceeds that shown by the curve for any day in the year the rate of draft may be any amount desired until the actual storage and the storage shown by the curve coincide. Thereafter, the rate of draft cannot safely exceed the rate for which the curve was computed. By developing curves at rates of draft of 450 500 550 and 585 000 000 gal. daily for the Catskill System, an excellent control diagram is made available for the engineer in charge of operation. During 1926 the rate of draft exceeded that shown by such curves, as the Catskill supply was about to be increased by the completion of the Gilboa Dam and the creation of the Schoharie Reservoir. A very low flow for the first ten months of 1926 with the Ashokan Reservoir almost empty demonstrated the advisability of adhering to a conservative rate of draft on a surface storage system.

There are a great many other supplies to which the application of the curves given by Mr. Sudler, and those developed earlier by Mr. Hazen,

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

would show that the supply usually drawn is in excess of that which would be reasonably considered a dependable supply for the community. Engineers should do what they can to educate the public to the fact that the supply obtained from a stream is not nearly equal to the average run-off and, in many cases, is equal only to a small fraction of the average run-off, if a dependable supply is to be secured. At present, in discussing the future supply of New York City, there are some who very strongly argue that the average flow of a stream is the basis on which one can determine the quantity of water safely available for the city's use.

In the Croton River the average flow for one period of 18 consecutive years was about 346 000 000 gal. daily, while for the following 18-year period the average stream flow was greater by about 100 000 000 gal. daily; and still there are those who say that it is possible to take the average flow in determining what may be estimated to be the safe yield.

The majority of people believe that, because water goes over the Croton Dam, there is an available supply which is thrown away, and that the city is being mulcted in some way of a part of the water supply it should have from the money expended to develop the Croton System. They will not believe the engineer who says that it is impossible to take all the average run-off of the Croton River, and still have enough left to supply the city when the run-off is below the average. It is not the most ignorant part of the population who believe this myth, but the more intelligent man.

Whether the majority of the citizens of New York will see the true picture of their water supply development is uncertain, but it is certain that a great deal of educational work should be done in eradicating from the public mind the idea that surface streams can furnish a supply equal to their average flow. Generally speaking, the so-called "safe yield" is certainly not a safe yield for even 98% of the years, and engineers should try to bring home the fact that the figures given do not represent a safe yield.

Richard H. Gould,* M. Am. Soc. C. E.—The paper† by Allen Hazen, M. Am. Soc. C. E., marked a distinct advance in the science of hydraulic calculations utilizing natural stream flow. The paper by Mr. Sudler represents an exhaustive study, utilizing many of the data and the method of analysis brought out in the earlier paper. It may really be considered an extension of the work started by Mr. Hazen. A study of this nature entails a tremendous amount of work and careful thought, usually much more than most engineers have an opportunity to spend for this purpose. The profession is, therefore, greatly indebted to Mr. Sudler for his results.

Because of the importance of the earlier work, it is gratifying to note that Mr. Sudler's independent study has shown that Mr. Hazen's seasonal storage curves can be depended on "in literally hundreds of instances". Mr. Sudler has suggested certain modifications in the higher rates of draft that are of interest. The speaker will confine his remarks principally to the sea-

* Prin. Asst. Engr., James H. Fuertes, New York, N. Y.

† "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

sonal storage curve for the lower rates of draft as applied to certain low stream flows.

A particular case in question is in connection with the yield of the impounding reservoir of the Cumberland, Md., water supply. These works, designed by James H. Fuertes, M. Am. Soc. C. E., were placed in operation in September, 1913. The collection works consist of a masonry dam about 70 ft. high, built across Evitts Creek, about 9 miles up stream from Cumberland. The reservoir formed is in the State of Pennsylvania, just above the Maryland State line. Water from this impounding reservoir, known as Lake Gordon, is filtered through a mechanical filter at the dam and flows by gravity in a 36-in. conduit to a distribution reservoir in Cumberland.

The water-shed area at the dam is 66 sq. miles, being about 18 miles long by 3 to 4 miles wide. The sides of the valley are steep, rising from 1 000 to 1 400 ft. above the creek bed, which is about 950 ft. above sea level at the site of the reservoir. The slopes of the hills are fairly well covered with timber, mostly of second growth. The soil is not deep, although there seems to be a considerable amount of fine sandstone on the water-shed. The reservoir has a water surface of about 5 900 000 sq. ft. when full and furnishes a storage, down to the lowest point at which gravity flow can be sustained, of about 1 000 000 000 gal.

A period of extreme dryness was experienced from June 17 to December 27, 1922, during which the reservoir was depleted to the extent of 810 000 000 gal. while sustaining an average draft of 7 000 000 gal. per day. The tax on the storage works was appreciably greater than had been anticipated in the design, and although it was possible to maintain a supply to the city at all times, the draft was reduced from a rate around 8 000 000 gal. daily at the start to about 4 200 000 gal. per day at the end. This was done without causing a great hardship, as Cumberland has a high water consumption per capita. Some anxiety was naturally felt as to the sufficiency of the works, which condition prompted further investigation as to the probable yield of the source.

The methods developed by Mr. Hazen seemed to furnish the best basis for this study, and if they are strictly applicable, indicate that the conditions of dryness along Evitts Creek in 1922 are among the most severe on record in the Eastern States. Based on the seasonal storage curves developed by Mr. Hazen and shown by Mr. Sudler (Fig. 6), and including the evaporation from the reservoir as part of the draft, the period in question corresponds to a 99% dry year. In deducing this figure, no allowance was made for "ground storage".

The 1922 period on Evitts Creek is unusual, but it is by no means certain that it corresponds exactly to a 99% year. The normal storage diagrams developed by Mr. Hazen are based on curves that are inclusive of the actual records of Eastern streams, with one or two exceptions. The most notable exception in the storages required in a 95% dry year is that computed for the lower developments of Tohickon Creek, near Philadelphia, Pa. These

lie above the normal inclusive curve selected. The Evitts Creek data are similar to those from Tohickon Creek in that relatively high storages are required in the small percentages of use. If the normal inclusive curve is raised to include the low developments of Tohickon Creek and the normal storage curves changed to correspond, the apparent degree of dryness on Evitts Creek would be appreciably less.

TABLE 4.—Flow of Evitts Creek at Lake Gordon, Cumberland, Md.

Month.	Total Monthly Stream Flow at Lake Gordon, in Million Gallons.									
	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.	1922.	1923.
January	2 554	2 046	1 411	1 822	170	1 028	513	509	1 400	486
February	1 284	3 025	1 497	3 131	3 751	479	537	705	2 097	1 070
March	2 540	1 395	2 937	3 731	2 718	2 025	2 619	1 923	4 000	974
April	2 811	647	2 453	1 915	2 733	977	1 111	504	2 507	1 728
May	1 331	1 150	1 587	976	660	4 215	1 203	1 299	1 563	889
June	455	2 810	2 089	1 929	281	466	2 257	503	365	238
July	293	666	1 734	335	130	559	376	720	128	219
August	139	2 773	540	366	172	262	672	1 002	84	1 080
September	138	673	614	169	159	133	433	914	46	217
October	168	133	529	846	153	408	140	183	71	84
November	196	493	326	431	235	668	406	1 193	57	229
December	365	1 034	572	211	860	1 173	1 118	1 807	161	1 018
Totals	12 274	17 545	16 289	15 862	12 022	12 393	11 385	11 262	12 479	8 182

TABLE 5.—Annual Flows at Lake Gordon, Cumberland, Md.
(Co-efficient of variation, annual flow = 0.216.)

Year.	Draft, in million gallons per day.	Run-Off.		Rainfall, in inches.
		In million gallons per day.	In inches on 66 sq. miles.	
1914	9.6	33.6	10.7	32.3
1915	8.5	48.0	15.2	45.4
1916	8.0	44.6	14.2	35.5
1917	8.7	43.5	13.8	33.0
1918	10.3	33.0	10.5	31.0*
1919	9.6	33.9	10.8	34.3
1920	8.8	31.1	9.9	32.9
1921	8.0	30.9	9.8	37.0
1922	7.7	34.2	10.8	21.8
1923	8.5	22.4	7.1	36.8
Average	8.77	35.52	11.28	34.30

* One month assumed.

The experiences which were had with the Evitts Creek development are not sufficient evidence to modify the normal storage curve in its lower portions. It may be noted, however, that the evidence here is quite similar to that brought out in the important records of Tohickon Creek. These considerations should be kept in mind in arriving at conclusions as to the probable recurrence of periods of similar dryness. Because of the geographical

location of Evitts Creek and the records of streams like Tohickon Creek, it is probably safer to assume that the 1922 dry period will recur oftener than once in a century. The value of the method of probabilities in classifying the 1922 dry period on Evitts Creek is clearly indicated.

The worth of the seasonal storage curve will increase from year to year as it is compared with additional records from different localities and found to be correct. The author has pointed out that the curve may require modification for use in some localities. Data that will establish the correctness or point out limitations in the use of the curve are of value. It is for this reason that the rather unusual records of the flows of Evitts Creek are submitted.

TABLE 6.—Weekly Record of Depletion in Lake Gordon, Cumberland, Md., in 1922.

(Average draft, 7 020 000 gal. per day; storage used, 12 300 000 gal. per sq. mile.)

Period ending.	Use, in Million Gallons.				Remarks.
	Draft.	Evaporation from lake.	From storage.	Stream flow.	
June 17, 1922 ..	....	....	....	....	Reservoir full.
" 24, " ..	54.8	5.7	15	45.5	
July 1, " ..	55.6	6.8	10	52.4	
" 8, " ..	55.3	5.0	25	35.3	
" 15, " ..	57.5	6.0	20	43.5	
" 22, " ..	56.9	6.4	30	33.3	
" 29, " ..	55.4	4.9	45	15.3	
August 5, " ..	59.1	4.5	10	53.6	
" 12, " ..	58.3	4.2	40	22.5	
" 19, " ..	61.2	5.1	45	21.3	
" 26, " ..	49.0	3.7	43	9.7	
September 2, " ..	66.0	3.6	42	27 6	
" 9, " ..	57.5	5.1	40	22.6	
" 16, " ..	66.6	4.0	50	20.6	
" 23, " ..	57.7	2.6	59	1.1	
" 30, " ..	54.6	2 0	56	0.6	
October 7, " ..	47.5	2.5	50	0	
" 14, " ..	38.8	1.5	20	19.8	
" 21, " ..	36.0	0.9	30	6.7	
" 28, " ..	40.9	0.7	20	21.6	
November 4, " ..	43.2	0.8	30	14.0	
" 11, " ..	39.8	0.5	20	20.3	
" 18, " ..	38.8	0.6	30	9.4	
" 25, " ..	44.2	0.3	20	24.5	
December 2, " ..	42.3	0.4	25	17.7	
" 9, " ..	39.2	0.4	8	31.6	
" 16, " ..	34.3	0 2	12	22.5	
" 23, " ..	29.5	0.6	5	25.1	
" 27, " ..	17.5	0.4	10	7.9	Day of maximum depletion, 193 from start.
February 14, 1923 ..	....	....	....	....	Reservoir full.
Totals.....	1 356.8	79.4	810	626.2	

Tables 4, 5, and 6 give the monthly and yearly records of the stream flow at Evitts Creek for a 10-year period, together with weekly records covering the period of maximum depletion of the reservoir in 1922. The records are subject to variation common to all hydraulic measurements of this nature, but are believed to be representative.

Mr. Arthur G. Fowler is the Superintendent in charge of the reservoir, filter plant, and pipe line, and it is to him that the speaker is indebted for the carefully kept records on which this interesting series of flows is based.

A. STREIFF,* M. AM. SOC. C. E. (by letter).—The author states: "The merits of the method of study here given [the Hazen method†] are generally unrecognized." The writer is unable to verify this statement, except by noting that the latest editions of the handbooks of hydrology do not give it in detail. In his practice, during the fourteen years that have elapsed since the first paper appeared, application on short-term records has shown certain inherent defects; but it must be said in all fairness that Mr. Hazen has warned against the use of these. Mr. Sudler, however, does not seem to concede these defects. While Mr. Hazen has expressed himself throughout with scientific caution, and merely states his belief in the greater accuracy of his method, Mr. Sudler takes the reliability thereof as a condition *sine qua non.*

This paper may be summarized as presenting a number of curves for rates of draft higher than those in the original paper, but it does not offer any additional proof nor investigate other rivers. Therefore, it cannot be said to have extended knowledge of the reliability of the method. Mr. Hazen's paper is one of the most important publications in the field of hydrology, but as time elapses and further studies are accumulating, certain inconsequences are becoming clearer. The findings of Mr. Hazen, far from representing probability methods applied to fortuitous numbers, might be classified as the establishment of certain simple laws of river flow based on statistics.

The writer, on the other hand, finds that these laws are not quite so simple, that wide departures therefrom may be expected. He does not admit that the computations given by Mr. Hazen, while unquestionably useful as rough approximations for just such problems as confronted Mr. Sudler, can be held to be more accurate than the ordinary method as used by the late Desmond Fitzgerald and by John R. Freeman, Past-Presidents, Am. Soc. C. E. In other words, Mr. Hazen, discovering certain simple relations in river flow, swings the pendulum too far in that direction.

The presentation of certain normal diagrams applicable on all rivers automatically dispels the notion that flow records are fortuitous numbers amenable to analysis by probability methods. As a matter of fact, little of the latter has really been used, and the whole procedure may be summed up in the following: The departures from the mean for a certain generally small number of years, n, plotted in the order of size are held to apply in the same proportion to any value of n.

In other words, the probability *a priori* is held to be equal to the probability *a posteriori.* This, in the theory of probabilities, is only true in case n is a very large number. If by plotting a comparatively small number of values in that manner certain normal diagrams result, this merely indicates the existence of certain laws applicable only to the comparatively short records

* Cons. Engr., Bangor Hydro-Elec. Co., Jackson, Mich.

† *Transactions,* Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

used; but it permits no conclusion as to "95% years" and the like, covering an infinite future. Besides, within the range of time covered, the departures of the individual rivers are quite large, and average values, applied on any one river, may be quite far from the truth.

The writer has advanced* a number of reasons for the belief that flow records are not unrelated numbers, but contain a large systematic element. This latter, on the other hand, is not as simple as expressed in Mr. Hazen's normal diagrams. It is not permissible to expect 300 or 1 000 flow-years to show the same proportions as a small flow record. In other words, flow records must be regarded as "historigrams",† and, as customary in statistics, all trends must be thoroughly removed before the remainder can be subjected to probability methods.

About twenty-six years ago the late Elmer Corthell, Past-President, Am. Soc. C. E., while visiting the jetties at the Hook of Holland, expressed to the writer his belief that all rivers are subject to certain regular fluctuations not unlike the tidal flows in estuaries. As a matter of fact, scientific literature at that time already contained many data tending to show that artificial 1 000-year records are contrary to established knowledge. During such long terms of years considerable changes do take place, which have a tendency to recur periodically. Perusal of the extensive literature of climatic changes will soon convince any one that artificial long records can not be considered seriously. Reference to the studies of many scientific investigators dispels the possibility of adopting artificial fortuitous records. Such arrangements simply do not exist in Nature, and engineers can ill ignore the findings of the scientists.

Long flow records are not available, but long historical records of high and low stages of rivers and lakes exist, and these, reaching far back into history, have been carefully studied. The record of the Nilometer at Cairo, Egypt, kept during the period of Arabian occupation for eight centuries and given by Aboul Mahâsin in the *Mémoires* de l'Institut d'Egypte, 1923,‡ shows plainly the periodic swings of long duration. The changes of the inland seas of Central Asia likewise indicate these long swings§ which relegate 1 000-year artificial records into the realms of fancy.

Also, for shorter terms, the existence of periodic variations has been studied carefully. The famous book of Professor Brückner, Geographer at the University of Berne, Switzerland, published thirty-seven years ago, gives elaborate statistics proving the existence of three variable climatic cycles the shortest of which, now called the "Brückner cycle", has a period of ± 35 years, but varying from 25 to 50 years.

Considering the still shorter periods of time, there are many studies covering small cycles of 2.5 to 3.5 years by Helland Hansen, Nansen, Bigelow,

* See p. 46.

† A graph which has time as abscissa and run-off as ordinate.

‡ "Ritmi cosmici nelle oscillazioni climatiche con speciale riferimento alla serie pluviometrica Patavina," G. del Valle, *Atti e Memorie* della R. Academia di Science, Lettere, ed Arti, in Padova, 1926, Vol. XLII.

§ "Le niveau des lacs de l'Asie centrale russe et les changements de climat," J. de Schokalsky, *Annales de Géographie*, Vol. 18, pp. 407–415.

Lockyer, Axel Wallén, Braak, Arctowski, Clough, and others. It is plain that fortuitous flow records can only be seriously considered after errors in the statistics of these investigators have been proved.

Neither can the amplitude of these systematic variations be neglected, especially for storage studies. The amplitude of the Brückner cycle in the San Diego, Calif., rainfall is more than 35% of the mean; the 11-year cycle was responsible for a 56% variation in the flow of the Huron River, in Michigan. These percentages are variable.

Such cycles cause the erratic fluctuations of some of Mr. Sudler's computations. For instance, he gives, for the coefficient of variation of the Croton River, for,

Ten	5-year means, maximum =	827%	of minimum
Five	10-year means, "	204%	" "
Two	20-year means, "	122%	" "

There is no direct evidence to show how this coefficient would have varied if twenty 20-year periods had been available. It is plain that Mr. Hazen's deduction based on the formula:

$$\frac{\text{Probable error in one term}}{\sqrt{\text{Number of terms}}}$$

is only applicable to fortuitous records, and does not hold for "historigrams".

It is significant that Mr. Hazen fully recognized the importance of the occurrence of cycles, and admitted the necessity of critically examining longer precipitation records of the region involved. Mr. Sudler, however, seems to regard the method to be more firmly established, stating, regarding short records, that "only one simple calculation is needed which may be based on a short-term flow record"; and, again, "to apply these to a particular stream for which only a few years of flow record are available", etc. In a case of the Huron River, in Michigan, where, due to the 11-year cycle, the four years, 1913-16, have a mean flow 56% higher than the four years, 1920-23, the determination of the storage constant from such a short record is evidently entirely false. Mr. Hazen, on the other hand, has specifically warned against the errors of short records. Also, within the year itself the existence of the cycle is evident, and, again, of not nearly as simple a nature as that expressed by Mr. Hazen's normal storage diagram.

The normal storage diagram is an average, from which wide departures may be expected. As a matter of fact, Mr. Hazen has already admitted this by stating that it represents climate. Far from indicating a probability of fortuitous nature, the normal storage diagram is a simple geometrical consequence of the law of yearly variation in river flow. Drawing a mass curve of monthly flows, it may be shown that within the year the low point of the curve usually occurs at a certain time, six to nine months later than the highest point. It is plain that when such a time difference between the two points is constant, the increments in storage for equal increments in draft will be equal.

This is a simple fact of plane geometry, entirely independent of any run-off characteristic except the time difference between the high and low points of the mass curve within the year. Likewise, the increasing dryness of the years tends to lengthen this distance, which, for each river, is quite a variable quantity from year to year. For rivers with widely different time differences between high and low points of the mass curves (such as found in the Pacific Northwest with its winter rains, where this time difference is apt to be three months), the normal storage diagram diverges still more; but for the individual rivers investigated by Mr. Hazen the differences are also considerable, and he arrives, by a process of smoothing and averaging, at a normal diagram that may be accepted as an approximate average, but hardly can be considered "more accurate than any but the longest flow records" of an individual stream. The normal storage diagram does not even apply to all rivers on the Atlantic Coast; the Penobscot River, for instance, requires a much greater seasonal storage.

Not only is it impossible to determine the storage constant from a short record with any measure of reliability, but if a reasonably long record is at hand it will take as much time to determine the constant as it does to determine storage. Furthermore, if, in agreement with scientific opinion, the view is held that the run-off record is a "historigram", the normal storage diagram loses the value ascribed to it, for the storage constant may be quite different for another phase of the Brückner cycle.

The computation of yearly storage based on the coefficient of variation also pre-supposes simpler laws of river flow than really exist. It is to be regretted that for the departures from the mean the term, "variation", is used, for, in statistics, this term is also used to indicate the difference of consecutive terms, the departures from the mean being termed "deviation" (Goutereau ratio). To state the draft in terms of the deviation from the mean is a clever artifice to reduce all rivers to a relative basis; but the question is whether this purpose is really thereby attained. This fact is only proved by Mr. Hazen* by allowing a large degree of approximation.

Beyond the length of one year the amount of storage is again determined by the presence of cycles in the record. The depletion diagram is a mass curve, and thus the amplitude of the longer cycles is enhanced, and the shorter suppressed, in amplitude. The most prominent cycle present in the length of the records used is the 11-year cycle and this is, therefore, the largest single factor in determining the annual storage; but there are others. These, combined, render the annual storage curve as dependent on many causes as the seasonal storage curve. Again, proportionality of increments of draft and storage only occurs if the distance between high and low in the mass curve is constant, and this occurs in the annual storage curve in discontinuous fashion; the control switches from the smaller cycles to the 11-year cycle and from this to the Brückner cycle, just as with increased draft the control switches from the seasonal to the annual curve. This would become clear to any one

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1595, Plate XXXVI.

who would plot the mass curve of the Tennessee River, for instance, at Chattanooga, Tenn., one of the longest records available in the United States.

Mr. Hazen then adopts a quasi-fortuitous arrangement by taking various records and placing these in succession regardless of their place in the scale of time. Mr. Sudler reduces the systematic features still more by the adoption of merely a certain degree of skew in selecting a certain duration curve, and building from it a storage diagram. The coefficient of variation does not completely describe the habits of the river, for $\sqrt{\frac{\Sigma \Delta^2}{n}}$ may be constant for entirely different groupings of Δ. This precludes reduction to the same relative basis by the coefficient of variation, for the storage required is, for the same coefficient, also dependent on the degree of skew.

Neither of the two methods appears to be admissible, as soon as one accepts the fact that the flow record is a "historigram". Accepting a fortuity theory the method of Mr. Sudler would seem to be more consistent, as Mr. Hazen preserves the records, arbitrarily chosen as to length, intact, but joins them in fortuitous fashion in the scale of time. It seems that for most streams a short record exists sufficient—according to Mr. Sudler—to determine the coefficient of variation; but is it to be concluded that the expensive stream gaugings of the U. S. Geological Survey may as well be discontinued, as the values are considered entirely fortuitous anyway? A perplexing task confronting the supporters of the fortuity theory is to point out errors in the statistics and conclusions of Professor Brückner's work,* wherein the climatic cycle is derived which changed, for instance, the mean flow of the Tennessee River 30% in the period, 1882-92, over the mean flow of the next 12 years.

As the matter stands, Mr. Hazen, while fully recognizing the influence of the cycles, considers these of minor importance. This belief the writer does not share. One instance of a large error was pointed out by Mr. F. B. Marsh.† The writer might mention the Huron River, showing a difference of 5.6 times the probable error in two 4-year periods, four years apart; the Tennessee, showing 2.5 times the probable error in one 10-year period; and the Penobscot, 4.3 times the probable error between two consecutive 4-year periods. The averaging of probable errors is no criterion, for, after all, the statistics are applied to individual cases.

The determination of the total storage curves by Messrs. Hazen and Sudler seems to the writer wholly inadmissible. The addition of the annual to the seasonal or monthly storage—on a somewhat different basis by both authors—can not be admitted, as they are not simultaneous values. The maximum depletion in annual storage occurs not necessarily (in fact, seldom) in the year of minimum flow. Neither does the maximum seasonal storage, although as Mr. Sudler's Fig. 10 shows, it follows the degree of dryness in a general way. The two are not related, and may occur in entirely different years. It should not be overlooked that the depletion diagram is a mass

* "Klimaschwankungen seit 1700".

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1644.

curve, hence it lags 90° in phase behind the hydrograph, the dryest year not occurring at the maximum depletion. Referring again to Mr. Hazen's Plate XXXVI,* the Croton River shows a maximum depletion for what might be estimated as a 70% dry year; taking the mean curve of Mr. Sudler's Fig. 10 the 70% seasonal and 95% annual should be added, and not as in Mr. Hazen's Table 15,† both 95% values. This is an important point which introduces errors in both Mr. Sudler's and Mr. Hazen's total storage curves, and which again illustrates the difficulty of attempting to eliminate the time as a dimension of the flow record.

Finally, if interpreting Mr. Sudler's Fig. 13 correctly, the writer finds very considerable differences in the amounts of annual storage between his and Mr. Hazen's diagrams:

Draft	= 1 —	0.4	*C*,	80%	dry year,	116%	difference in storage
"	"	90%	"	"	,	55%	"	"
"	"	95%	"	"	,	40%	"	"
"	"	98%	"	"	,	24%	"	"
"	"	99%	"	"	,	24%	"	"

It is, of course, quite important whether a reservoir should be, say, 40% larger or smaller; such divergent results, if correctly arrived at by the writer, can be characterized as gross approximations only. Since Mr. Sudler uses Mr. Hazen's values for the seasonal storage, the diagrams of both are alike.

Mr. Hazen considers the use of a mass curve giving only the accumulated departures from the mean, a contribution of real importance to the methods of making storage calculations. It is indeed surprising that it has not been in universal use, for it was published long ago.‡ Such mass curves show the succession of groups of dry and wet years very plainly and are well suited to demonstrate the periodic structure of river flow.

The writer has freely criticized the paper and the method on which it rests, believing the subject of great importance and deserving careful consideration. He is not prepared to follow Mr. Sudler and Mr. Hazen in their attempts at such a sweeping simplification of the tangled skein of terrestrial phenomena as a normal storage diagram for all rivers and all time expresses; but he pays tribute to the great amount of thought and labor involved in these investigations, which undoubtedly have delineated many questions on river flow heretofore not discussed.

Sigurd Eliassen,§ Assoc. M. Am. Soc. C. E. (by letter).—One of the notable ideas in this paper on the extension of Mr. Hazen's method for analyz-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1595.

† *Loc. cit.*, p. 1597.

‡ *Water Supply Paper No. 279*, U. S. Geological Survey, Pl. X, p. 197, published in 1914, is just such a mass curve. The authors of this paper are H. K. Barrows and C. C. Babb, Members, Am. Soc. C. E. The writer has used such mass curves for many years, and published such curves in the *Monthly Weather Review*, July, 1926. *Bulletin No. 5*, Dept. of Public Works, State of California, an Appendix to the 1923 Report on the Water Resources of California, contains a great many mass curves of California streams in that form.

§ Engr.-in-Chg., Survey Dept., Chihli River Comm., Tientsin, China.

ing stream flow in connection with storage requirements is the author's manner of supplementing the data by means of chance drawings.

The statistician, with records at his disposal, such as would make an hydraulic engineer's mouth water, sneers at the engineer's attempt, in extrapolation from short-term records, to use the method of probabilities as a tool for guiding his judgment. "Keep within the limits of your data", says the statistician; but, unfortunately, that is just what the engineer cannot do. He must try as well as possible to argue beyond. Duration curves, Mr. Hazen's probability paper,* and, of late, Mr. Goodrich's skew probability paper,† are all devices for making the extrapolation safer. To this is now added the author's idea of "creating" an hydrological universe from a duration curve constructed from short-term records, and of making chance drawings from this universe.

The idea has considerable merit but the writer questions the author's method of drawing. It is not clear why the drawn card was left out after each drawing. Surely the tendency by not putting the card back would be sooner or later to force an unduly prolonged dry or wet period. Why not put the card back again, thus drawing from a universe of constant proportions? Even this is perhaps not strictly correct, as periodic changes in the solar radiation influence the conditions. However, as long as the universe is made up from a duration curve which in itself somewhat reflects the effect of these periods, no very serious error should result on this account as long as the water-shed is small. For large water-sheds the tendency to well marked periodicity in run-off may become an important factor.

What reliance can generally be put on a short-term record? Avoiding dubious mathematics the writer has taken a duration curve of maximum flood discharges of the Yung Ting River in China for the 15-year period, 1912 to 1926. Assuming it to reflect within a small margin of error the actual condition, he has made a universe of 100 records between the limits 6 000 cu. m. per sec. to 240 cu. m. per sec. Then 2 500 chance drawings were made keeping the conditions constant. The resulting "duration" curve is shown in Fig. 37 (*a*). It lies very close to the original curve which, of course, was to be expected. On Fig. 37 (*a*) are also drawn "duration" curves for the lowest and the highest 25-year period (25-year periods were used in order not to obtain absurd results). Two other chance series were also made from the same universe, namely, by choosing, at random, five 25-year periods and one 125-year period and dividing the latter into five consecutive 25-year periods. The "duration" curves were constructed for these periods and are shown in Fig. 37 (*b*) and Fig. 37 (*c*).

The object in constructing these curves is to show a few of the endless variety of universes it may be possible to obtain by slicing out short periods from a long-term universe. At first glance, even a 25-year record does not seem to offer much of a basis for building up a universe that will represent the actual conditions. However, by further considering the case it is per-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† See p. 1.

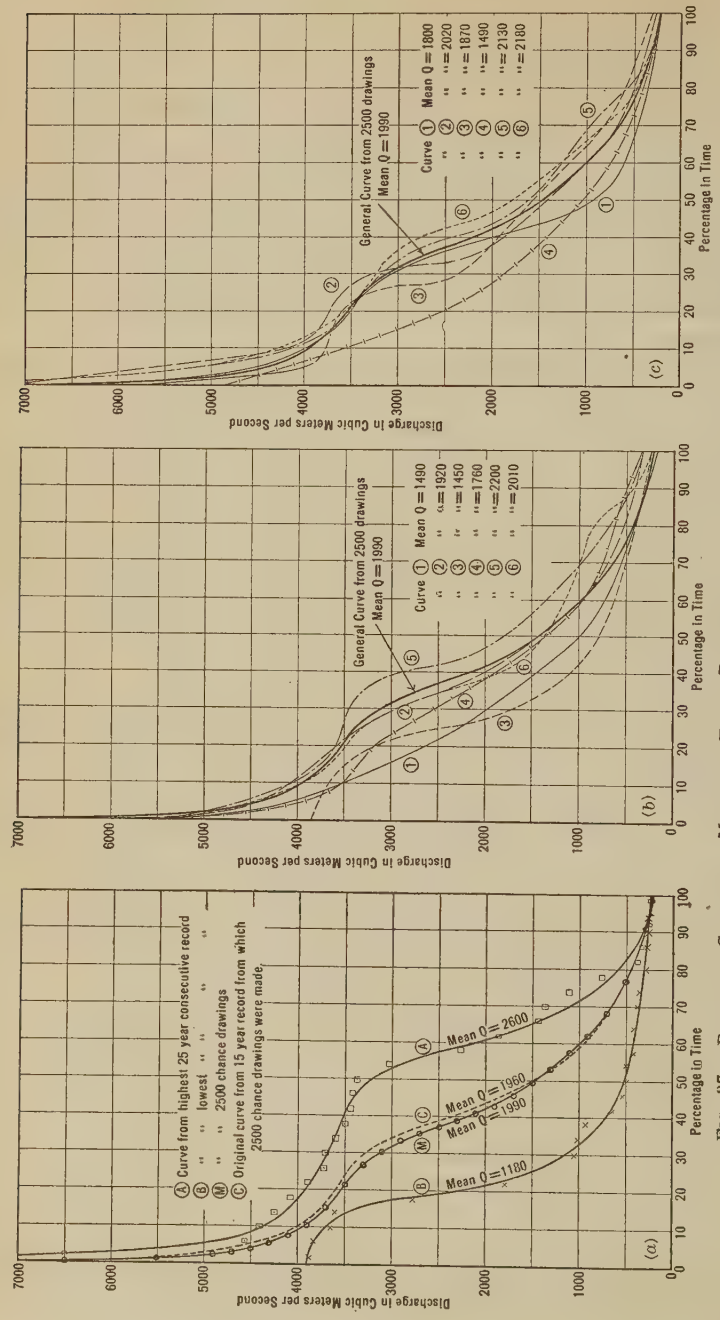
Discharge in Cubic Meters per Second
Percentage in Time
(a)
(A) Curve from highest 25 year consecutive record
(B) " " lowest " " "
(M) " " 2500 chance drawings
(C) Original curve from 15 year record from which 2500 chance drawings were made.
Mean Q = 2600
Mean Q = 1960
Mean Q = 1990
Mean Q = 1180
(b)
General Curve from 2500 drawings
Mean Q = 1990
Curve ① Mean Q = 1490
" ② " " = 1920
" ③ " " = 1450
" ④ " " = 1760
" ⑤ " " = 2200
" ⑥ " " = 2010
(c)
General Curve from 2500 drawings
Mean Q = 1990
Curve ① Mean Q = 1800
" ② " " = 2020
" ③ " " = 1870
" ④ " " = 1490
" ⑤ " " = 2130
" ⑥ " " = 2180

FIG. 37.—DURATION CURVES OF MAXIMUM FLOOD DISCHARGES, YUNG TING RIVER, IN CHINA, 1912–1926.

haps not as bad as it looks. It is evident that the chances are very small indeed that one should just happen to strike some of the wettest or dryest 25-year periods, which a very long-term universe is likely to produce. They would surely be noticeable periods much commented on by the population of the district. On the other hand the chances are very largely in favor of the period from which the universe is made up being rather close to the average condition, and if the records are used for the purposes of determining the longest sequence of dry or wet years a short-term record, in most cases, will serve well for building up a universe from which chance drawings can be made as suggested by the author. For extrapolation of such happenings as maximum floods the method may not be so well adapted.

The author pre-supposes that hydrological events are chance happenings, but this is true only to a certain extent. The total visible and invisible water content on the earth and in its enveloping atmosphere may be regarded as constant at least for periods of several hundreds of years at a time. Small variations of a returning character take place; or, putting it another way, the evaporation-precipitation cycle is fluctuating. The main factor bringing about changes in this cycle is variation in the amount of heat that reaches the earth from the sun. The other factors, such as the escape of small quantities of water vapor into space, the variations in available water content due to changes in vegetation and to human activities, and also leakage into the interior of the earth and escapes therefrom due to volcanic activities, may be regarded as having so slight an influence as to be negligible.

Changes in solar radiation are correlated with outbreaks of faculæ on the sun, which again are correlated with the appearance of sunspots. Based on accurate solar radiation measurement some meteorologists are now ready to assert that increases in solar radiation are accompanied by increased atmospheric activities on the earth and are directly the causes of anti-cyclonic and cyclonic storms.* The atmospheric pressure during such periods becomes lower in the tropics and higher in the temperate zones and again lower around the poles, resulting in increased rainfall in the tropics and often increased aridity in the temperate zones, although in the latter region rainfall may be intensified in certain localities depending on the paths of cyclones.

Taking the earth as a whole the sum total of variation in climate is relatively slight from year to year, even during years of maximum faculæ; yet the variation can be considerable for a small part of the earth's surface, such as a water-shed, and it is here that the matter of chance should enter. The tendency of rainfall to concentrate erratically; the striking, non-striking, or partial striking of small water-sheds by rain-bearing cyclonic "lows" having their origin several thousand miles away, certainly belong to the realm of chance even in years when the solar radiation is higher than ordinary. In such years, however, the chances are increased that the rainfall or the aridity, as the case may be, will be intensified due to the greater number and greater intensity of "lows" and "highs."

* "The World Weather," by H. Clayton.

A serious outbreak of sun faculæ usually lasts for a few months. Its effect on the atmospheric activity on the earth will be felt about every 14 days, or when the faculæ due to the sun's rotation appear on the side of the sun facing the earth, the sun's rotation having a period of about 27 days. It becomes a matter of chance, then, if for a particular area the outbreaks will coincide with the wettest or dryest period, or produce negative results.

Also, regions lying in the paths of both cyclonic "lows" and "highs" are generally more exposed to variations in climatic conditions than regions lying away from the general paths of these storms. North China, for instance, lies directly in the paths of the Siberian anti-cyclones and cyclones. These are most prevalent during the winter and early spring. During July and August the Pacific typhoons often cross it. Hence, aridity during the winter and spring and intense rainfall or drought during the summer are the prevalent conditions, and maximum faculæ periods seem to coincide either with the maximum droughts or with maximum flood periods.

If some one having solar radiation data at his disposal would examine the possibility of seasonal periodicity in connection with the 3.6 and 11.3-year periodicity, he could perhaps find an approximate law underlying variations in rainfall and run-off even for small water-sheds. At present, it seems that effects of cyclic changes in solar radiation on stream flow can only be shown for very large water-sheds, such as the Mississippi River and the Great Lakes. The laws governing the flow of smaller rivers are very obscure, but can perhaps be discovered when the basic facts governing the "disturbing" or, rather, the "order-producing", influences are unearthed.

Variations in solar radiation are now being intensively studied by means of bolometer measurements, the latest observatory being established in South Africa by the Smithsonian Institute.

Theodore Hatch,* Esq. (by letter).—At present, hydrological records are available for only comparatively short periods of years. The maximum or minimum experience shown in such records, therefore, cannot be used directly in the design of structures that must serve over longer periods of time. Statistical analysis by probability methods can be of assistance in determining from a study of the shorter records the most probable conditions that will occur during longer periods of time. The author has made use of such methods. Based on Foster's conclusions that a Type III frequency curve, with $C\ S = 0.6$, best fits the records of stream flow in New York State, he has used the theoretical curve to build up an artificial record extending over 1 000 years for the purpose of studying long-time variations in stream flow and resulting storage requirements.

The idea of utilizing an artificial record in this way is a fascinating one. It is the purpose of this discussion, however, to point out the difficulty of securing statistically significant results and the limitations of the findings based on an analysis of this type.

At the outset it should be questioned whether the statistical arguments for the use of a Type III frequency curve are sufficiently strong, and,

* Instr. in San. Eng., Harvard Univ., Cambridge, Mass.

following this, whether a chance drawing of artificial values, as undertaken by the author, can be made to yield significant results and reproduce a true picture of stream flow.

As to the first, analysis of the longest stream-flow record in the United States, that of the Croton River, shows characteristics very different from a Type III curve. Its various parameters are given in Table 7.

TABLE 7.—Statistical Parameters of the Croton River.

Length of record, in years	59
Mean annual flow, in million gallons per day per square mile	1.065 ± 0.0214
Standard deviation	0.2428 ± 0.0151
Coefficient of variation	0.2280 ± 0.0142
Value of β_1	0.1696 ± 0.1053
Value of β_2	2.2148 ± 0.1940
Value of κ	−0.0664 ± 0.0446
Coefficient of skewness	0.4118

The Pearsonian criterion, κ, indicates Type I as the curve of best fit, that is, an asymmetrical distribution limited in both directions. It will be observed, however, that the probable errors of β_1, β_2, and κ are so large as to make the parameters themselves unreliable as criteria of the type of distribution; hence, any conclusions drawn from them remain uncertain. Indeed, the departures of β_1, β_2, and κ from their values in the normal frequency curve, 0, 3.0, and 0, respectively, are not as great as might be obtained from errors common to random sampling; that is, a normal frequency curve may be taken as the curve of best fit. Further analysis of the data by Pearson's method of finding the probable frequency type,* however, indicates high probability of Type I being the curve of best fit.

To the writer this seems to be a logical conclusion, which is supported by the following considerations. Examinations of frequency distributions covering wide fields of hydrological phenomena indicate clearly the presence of skewness, and *a priori* reasoning leads to the conclusion that upper and lower limits do exist. Thus, it seems logical that the frequency curve of run-off should be limited at both ends when one considers that the casual agents of rainfall, evaporation, wind velocity and direction, temperature, and the other factors entering into run-off do have upper limits as well as the lower limit of zero. The extreme variations occur when these casual agents combine in maximum or minimum force.† That this reasoning is not borne out in every case of hydrological analysis is due to the fact that the records are not available over long enough periods of time. This fact has been recognized by Foster in connection with the accuracy of the coefficient of skewness and of the fourth moment,‡ but it is also significant in connection with the second moment. Thus, sixty observations are considered necessary by Leavitt and Gowan to make the standard deviation stable.§ In

* "Tables for Statisticians and Biometricians," p. lxviii.

† The rainfall at Boston, Mass., over a period of 96 years shows a Type I distribution.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 165.

§ "A Contribution of Statistics to Practical Mortar Testing," by Leavitt and Gowan, *Bulletin No. 12*, Maine Technology Experiment Station.

the absence of long-time records the author's use of a Type III curve, while perhaps not conforming to hydrological evidence, will, on the whole, lead to higher and, therefore, safer values than the use of a Type I curve.

The second question relating to statistical procedure arises in connection with the author's manner of using the theoretical stream flow record chosen. He states:

"In the present instance fifty annual run-off values were selected and put on cards, which were shuffled and drawn one by one until the whole fifty were used. Twenty such drawings were made, thus providing a 1 000-year artificial record."

The fifty values were determined by dividing the theoretical duration curve into 2% frequency intervals and reading the values at the midpoints, 1%, 3%, 5%, etc.* The duration curve for $C\ S = 0.6$ and $C\ V = 0.16$, divided in this manner, is shown in Fig. 38.

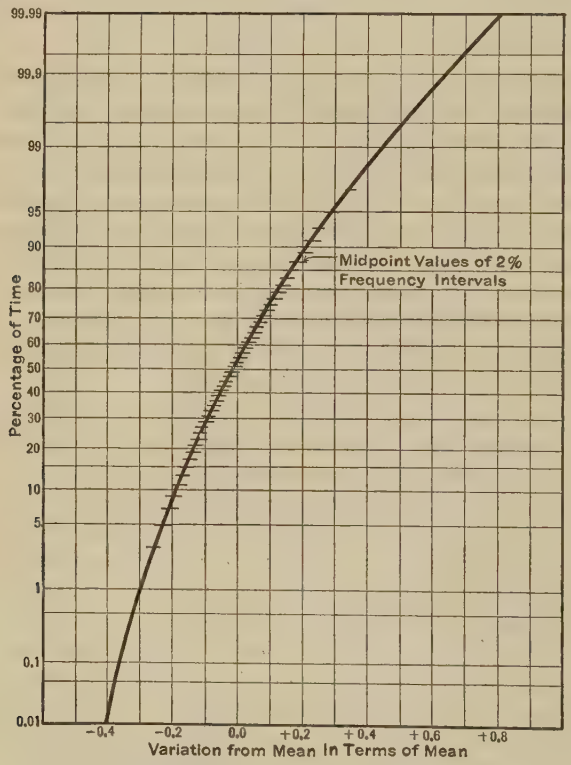

FIG. 38.—THEORETICAL RUN-OFF SUMMATION CURVE.
$C\ S = 0.6$; $C\ V = 0.16$.

This procedure gives unequal class-intervals and is counter to a general rule of statistical methods, which requires that class-intervals should be equal in order to make possible the comparison of frequencies of different values,†

* This information was given by Mr. Sudler in a personal communication, February 12, 1927.

† "Theory of Statistics," by G. Udny Yule, p. 76.

an object particularly sought in this case. As used by the author the varying class-intervals each have a frequency of 20 per 1 000 and the resulting histogram shows a rectangular distribution (see Fig. 39) in which the maximum and minimum values are not those that occur once in 1 000 years but only once in 100 years. By taking his fifty values twenty times the author does not actually construct a record including the 1 000-year experience. He obtains merely twenty of the 50! possible combinations of values that have a 100-year maximum and minimum limit.

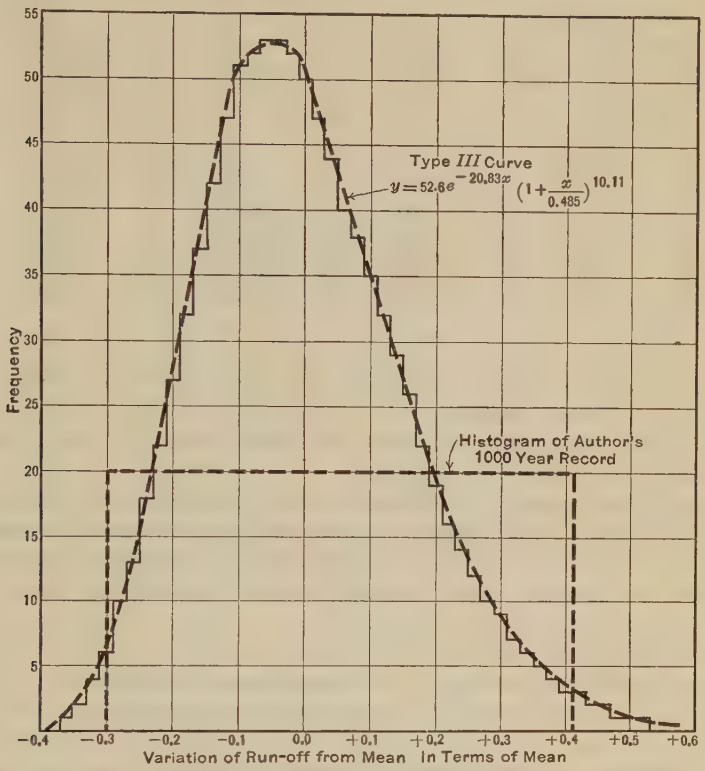

FIG 39.—THEORETICAL RUN-OFF FREQUENCY CURVE AND 1 000-YEAR HISTOGRAM. $C\,S = 0.6$; $C\,V = 0.16$.

The writer has used a different means for constructing and utilizing an artificial 1 000-year record containing all the variations incident to the entire period and has subjected the results obtained to analysis for the purpose of determining their significance. To meet the purposes of this discussion the theoretical Type III frequency curve with $C\,S = 0.6$, $C\,V = 0.16$, and $N = 1\,000$ was made the basis of the writer's record. The equation of this curve* was found to be,

$$y = 52.6\ e^{-20.83\,x}\left(1 + \frac{x}{0.485}\right)^{10.11}$$

A histogram was erected on the theoretical frequency curve in such a way

* "Frequency Curves and Correlation," by W. Palin Elderton, p. 65.

that the frequencies obtained were integers. The histogram obtained is shown superimposed on the theoretical 1 000-year frequency curve in Fig. 39. Table 8 gives the frequencies as determined from the histogram in Fig. 39.

TABLE 8.—Frequency Distribution of Run-Off Data Artificial 1 000-Year Record.

(From Histogram, Fig. 39.)

Run-off variation, in terms of mean.	Frequency.	Run-off variation, in terms of mean.	Frequency.	Run-off variation, in terms of mean.	Frequency.	Run-off variation, in terms of mean.	Frequency.	Run-off variation, in terms of mean.	Frequency.
−0.36	1	−0.18	32	−0.00	50	+0.18	22	+0.36	5
−0.34	2	−0.16	37	+0.02	47	+0.20	19	+0.38	4
−0.32	4	−0.14	42	+0.04	43	+0.22	16	+0.40	4
−0.30	6	−0.12	47	+0.06	40	+0.24	14	+0.42	3
−0.28	10	−0.10	51	+0.08	38	+0.26	12	+0.44	2
−0.26	13	−0.08	52	+0.10	35	+0.28	10	+0.46	2
−0.24	18	−0.06	53	+0.12	32	+0.30	9	+0.48	1
−0.22	22	−0.04	53	+0.14	29	+0.32	7	+0.50	1
−0.20	27	−0.02	52	+0.16	26	+0.34	6	+0.52	1

Since it would be difficult to mix 1 000 cards thoroughly and, consequently, to obtain a pure chance sequence, the 1 000 values were written on lima beans. Perfectly round objects of equal diameters would have been better, but the beans were the best substitutes available.

The beans were drawn in two different ways. In the first case pure chance drawings were made. This method ignored the phenomenon of cycles. In the second case a cyclical effect was introduced by drawing first six negative values, then five positive values, then six negative values, and so on. The effect of this was to give 11-year cycles.

Two drawings (No. 1 and No. 2) were made in each case. From the hydrographs thus constructed curves of cumulative annual depletion for a draft of 1.0 times the mean annual flow were computed and plotted in Fig. 40. For purposes of comparison the author's curve of cumulative annual depletion for a draft of 1.0 times the mean annual flow (Fig. 2) was also plotted.

It is evident that these curves are not in close agreement. A comparison of the various parameters is made in Table 9.

TABLE 9.—Statistical Parameters of Curves of Cumulative Depletions.

	Chance Drawing.		Cyclical Drawing.	
	No. 1.	No. 2.	No. 1.	No. 2.
Mean	3.40	1.78	2.17	3.00
C_V	0.64	0.76	0.28	0.52
C_S	0.49	1.02	0.49	0.02
κ	−0 1225	−0.0317	−0.0819	−0.0002
Type curve	H	H	H	H
95% value	6.1	4.5	3.5	5.3

The reason for this lack of agreement among the curves lies in the fact that there are a great many possible combinations of the values, which give as many different curves of cumulative annual depletion. The number of possible combinations is naturally less in the case of cyclical drawings than with pure chance drawings. With both methods of drawing, minimum conditions of depletion are given when the values arrange themselves so as to just balance each other. Under such conditions cumulative depletions are

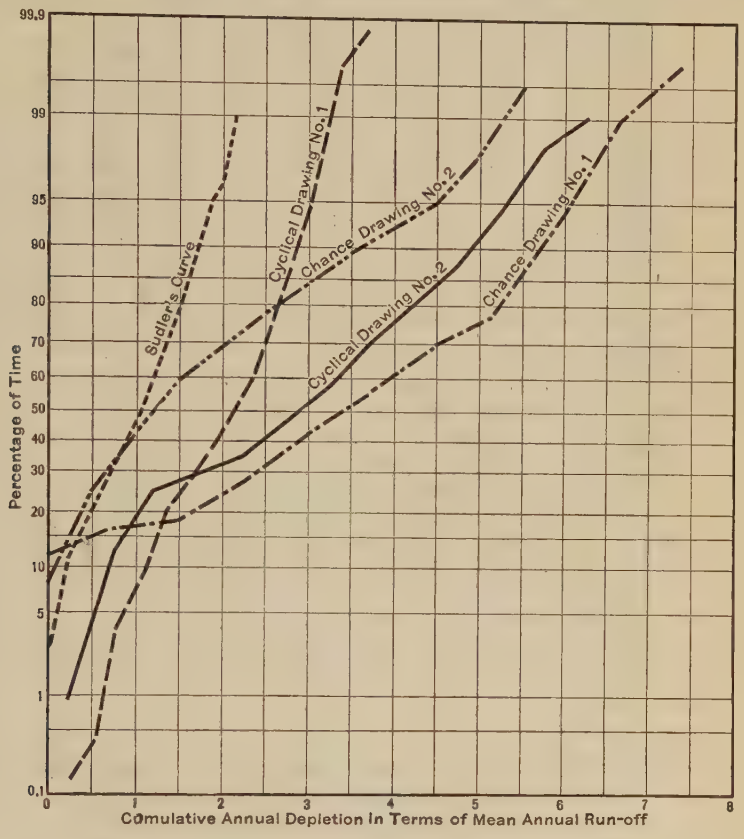

FIG. 40.—DURATION OF CUMULATIVE ANNUAL DEPLETION. DRAFT = 1.01.

approximately zero throughout the entire period. The curve of maximum depletion from the chance drawing is given when the values occur in order of magnitude beginning with the dryest year or largest negative value. The frequency distribution obtained is J-shaped and has a maximum value equal to the sum of all negative variations. With the cyclical drawing the curve of maximum depletion is given when the six highest negative values are followed by the five lowest positive values, and these are followed by the second six highest negative, etc. Curves of maximum and minimum depletions for both chance and cyclical drawings are shown in Fig. 41.

The curve of depletion for a particular combination of the 1 000 values will fall somewhere within the limits shown. It becomes evident, therefore, that one chance drawing of the 1 000 values as undertaken by the author is not dependable. If any use is to be made of such a method as this the most probable curve must be determined. Most likely it would be easier to do this experimentally by determining the depletion curves from a large number

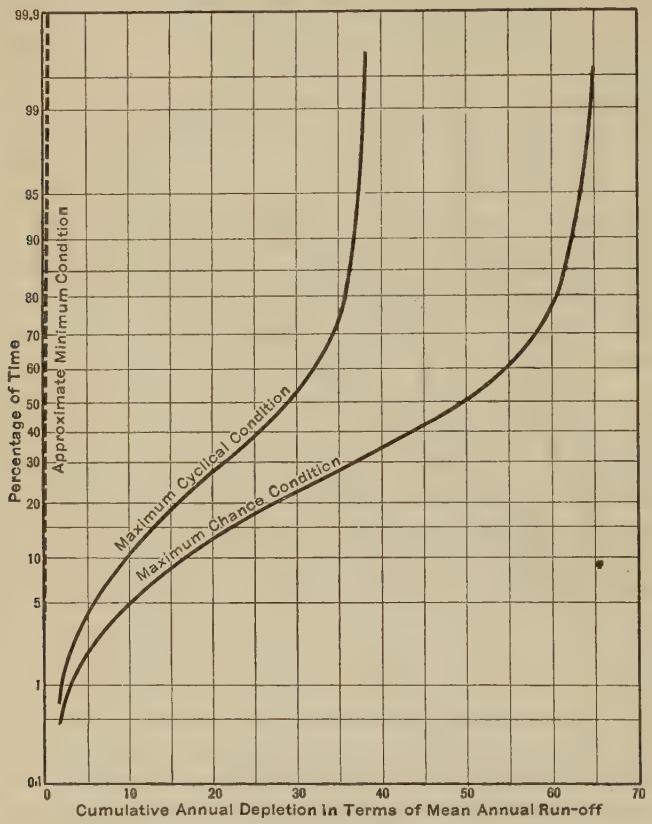

FIG. 41.—MAXIMUM AND MINIMUM DEPLETIONS FOR DRAFT = 1.0, CHANCE AND CYCLICAL HYDROGRAPHS.

of drawings rather than to set up the necessary mathematical equations. It is seen that the cyclical effect introduced by the writer reduced the number of possible combinations and narrowed the limits within which the curve might range. Since there is strong evidence of cyclical variation in hydrological data it would be desirable to introduce a cyclical effect in a study such as that of the author. There are several methods of doing this, however, and the best method will be difficult of determination.

L. T. GUY,* Assoc. M. AM. SOC. C. E. (by letter).—This excellent paper clearly shows the result of much thought and rather laborious calculation.

* Asst. to Civ. Engr., State Electricity Comm., Melbourne, Victoria, Australia.

There are, however, several points on which the writer's experience, in charge of hydro-electric investigations in the State of Victoria, Australia, would indicate that the conclusions reached by the author are open to question.

Mr. Sudler's method of calculating an artificial record covering an extended period such as 1 000 years, appears to involve the following assumptions:

(1) That no annual discharges will occur in the future that are not within the range of past short-period records.
(2) That these past values of discharge may occur in any chance order, so that sooner or later the worst possible sequence of dry years will occur.

Assumption (1) appears obviously incorrect. Assumption (2) is at least open to question, but if it is granted, the most logical method would be to take at once the worst sequence of dry years and from it to compute storage requirements—which would reduce the question to an absurdity. In any case, the procedure of selecting values by some chance method for 1 000 years appears to involve a considerable amount of laborious work without in any way increasing the accuracy of the results.

The basis of most of the curves in the paper is the "mean annual flow" and the results obtained are in terms of this "mean annual flow", but no method is suggested for obtaining the correct value of such "mean annual flow" from short-period records, even if the other factors, such as $C\,V$ and $C\,S$, can be obtained with reasonable accuracy from such records.

A typical illustration of the difficulties involved in this determination is furnished by a Victorian stream, the Mitta-Mitta River. This river drains an area of about 2 500 sq. miles, much of which is at a high altitude, and the discharge ranges from 25 000 sec-ft., or more, to 85 sec-ft. in an extreme drought. Stream gaugings are available for a period of 40 years, and a plot of the theoretical duration curve* shows close agreement with the points calculated from the actual discharge records, with $C\,V = 0.48$ and $C\,S = 1.4$, the mean annual discharge over the whole period being 1 160 000 acre-ft. per year.

Assuming that discharges were available only for the first period of 10 years on record (1886-95), the corresponding values obtained would be as follows: For a mean annual discharge of 1 410 000 acre-ft., $C\,V = 0.31$, and $C\,S = -0.13$.

Further, the mean annual discharge for the ensuing period of 10 years, that is, from 1896-1905, was only 840 000 acre-ft. per year. An assumption of mean annual flow on this latter 10-year period would have been nearly 30% too low in this case, while the result from the previous 10-year period would have been more than 20% too high.

The careful study and the curves evolved by Mr. Sudler are most useful in leading to a right attitude of mind in the interpretation of stream-flow data, but the writer considers that there is no satisfactory method of extending a short-period record to cover a much longer period without grave risks of large

* Calculated on the lines followed by H. Alden Foster, Assoc. M. Am. Soc. C. E., in his paper entitled "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

errors, and that all such theoretical calculations should be interpreted with caution as there is always the risk that a sense of false security may be created by their use.

Mr. Sudler's method of plotting mass curves is interesting and is convenient in that paper of only ordinary size is required. Another method, giving the same result, has been in use by the writer for some time and appears to have some slight advantage over that suggested by the author. The curves are plotted on ordinary square ruled paper with the actual summation of river discharges, but with the base line for the vertical scale drawn on a slope corresponding approximately with the slope of the mean discharge. In Mr. Sudler's method, the value of the mean annual discharge may vary with a longer period of record. In the writer's method this difficulty is avoided, and storage requirements may be read off directly in acre-feet, or any other desired units.

According to Mr. Sudler (page 629) the value of $C\ V$ must be less than one-half the value of $C\ S$. Some streams do not appear to support this statement,* including those in Australia, for which the writer has records up to a period of ten years.

Charles E. Sudler,† M. Am. Soc. C. E. (by letter).—Some of the points brought out in the discussions are beyond the intended scope of the paper, but are pertinent and valuable contributions to the subject, as, for example, Mr. Brush's criticism of the term, "safe yield". The expression, "dependable" flow, or yield, is open to much the same objection, as the degree of dependability is seldom defined.

Mr. Brush and Mr. Sargent both describe the study of existing flow records in order to plot operating curves by which a nearly ideal use of storage can be obtained while still insuring sufficient reserve to maintain the computed dependable minimum during dry periods. Such diagrams are very useful in regulation for power purposes especially, but must be based on careful analysis, particularly where a large amount of annual storage is provided. In this case a portion of the storage must be retained for use only during a period of one or more years having annual discharge less than the assumed draft.

Mr. Hazen suggests that the discrepancies between the storage values of his and the writer's diagrams may be due to definition. Comparison of the methods of study reveals that the estimated annual deficiencies, both singly and cumulatively, do not differ greatly. Those computed by the writer from the artificial record appear to be slightly, but only slightly, smaller than those of Mr. Hazen based on the combined flow records of a number of streams. Although the deficiencies are nearly identical, the required annual storages differ greatly, because the two methods of analysis of deficiencies are entirely dissimilar.

The writer's view is that if storage is to be provided for, say, 90% of the years, this percentage should apply equally well to long or to short periods.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 171, Table 5.

† Tarrytown, N. Y.

Actual length of period in years should have no effect on the computation. Mr. Hazen's procedure was to compute all the depletions of a reservoir of infinite capacity. These depletions were plotted in order of magnitude and the storages were then read at points on the curve corresponding to the required percentages of years.

This method was tried, giving the curves shown on Fig. 2. On this diagram may be seen Mr. Hazen's curve for 0.95 draft, lying somewhat above the corresponding line as computed from the artificial record. For this extremely high draft the annual storage indicated on Mr. Hazen's curve is 0.5 mean annual flow, and that from the artificial record is about 10% less. This indicates that the summations of annual deficiencies as found in the artificial record are somewhat smaller than those of the combined actual flow records used by Mr. Hazen. The difference, however—only 10% in case of 0.95 draft—is not large, and it might be argued that, as shown by the comparisons in Fig. 2, the depletions obtained from the artificial record give results almost identical with those found by Mr. Hazen. On this diagram (Fig. 2) Mr. Hazen's curves for 0.95 and 0.90 drafts are plotted as broken lines, slightly above the solid lines as found from the artificial record.

Fig. 2, as stated in the paper, indicates the depletions obtained with a reservoir of infinite capacity, full at the start. This method of analysis appeared to be illogical and was abandoned. It is clear that in respect to annual storage a draft equal to mean flow can be maintained without storage during all those years when the flow exceeds the mean. For the streams considered, 46% of the years have annual flows exceeding the mean, therefore no annual storage is required for 46%-year regulation. Yet Fig. 2, computed by Mr. Hazen's method requires an annual storage for 1.0 draft beginning at about 4% of the years; and for 46% of the years it calls for an annual storage nearly equal to the entire flow of the mean year.

Table 1 was prepared to illustrate the marked difference in results obtained by the two methods of computing annual storage. This table has been extended as Table 10 in order to simplify comparison of the two methods. It will be noted that by the depletion method, with infinite storage, 0.69 storage is called for during 19 out of 20 years, and that 0.44 storage is called for in the eighth year, or for 40%-year regulation, whereas in actual operation this 0.44 storage suffices to maintain the draft for 19 of the 20 years. It will be further found, as is indicated in Fig. 3, that the amount of storage called for by the depletion method depends on the length of the record examined. Therefore, the depletion method is inadmissable from a logical standpoint if storages for certain percentage of the years are to be computed. 95%-year storage should be equally applicable to 19 out of 20 years, or 95 out of 100 years as long as there is no change in the stream flow characteristics. .

It is true that with 95% regulation and very high rates of draft occasional deficiencies of considerable magnitude may occur. If these must be avoided then the estimated draft for a given storage must be reduced, or more storage must be provided, as perhaps by using the 99%-year instead of the 95%-year diagram.

Regarding Mr. Hazen's mention of the results using 0.69 storage to maintain 0.9 draft on the Croton River, it may be noted that the deficiencies for 2 of the 4 years were 2% and 5%, respectively. These are small deficiencies which might easily have been eliminated in case of a longer or a shorter period, or of a slightly different mean, or CV, or storage constant. If they are ignored then the draft would have been maintained for 96% of the years as against the assumed 95 per cent. Although a deficiency of 22%, as found for one year in the case of the Croton, with 95%-year storage based on the writer's curves, is a serious matter, it is not unique. Mr. Gould, for example, mentions a reduction in the rate of water supply for Cumberland, Md., of nearly 50%, due to an exceptionally dry year. This is not meant to disparage Mr. Hazen's evident plea for conservative statements as to expected supply. In fact, the writer would go further, and advocate the use of the 99%, rather than the 95%-year flow, in water supply computations.

TABLE 10.—Comparison of Use of Limited Storage, and Depletion of Unlimited Storage.

Year.	Run-off.	Reservoir of Infinite Capacity.		Reservoir of 0.44 Capacity.	
		Depletion.	Depletion in order of magnitude.	Storage available.	Deficiency of flow.
1	1.20	0	0	0.44	0
2	0.74	0.26	0.23	0.18	0
3	0.82	0.44	0.26	0.0	0
4	1.13	0.31	0.31	0.13	0
5	0.93	0.38	0.38	0.06	0
6	0.69	0.69	0.40	0.0	0.25
7	1.16	0.53	0.40	0.16	0
8	0.91	0.62	0.44	0.07	0
9	0.99	0 63	0.45	0.06	0
10	0.94	0 69	0.48	0.0	0
11	1.29	0.40	0.53	0 29	0
12	0.85	0.55	0.54	0.14	0
18	1.01	0.54	0.55	0.15	0
14	1.09	0.45	0 59	0.24	0
15	1.22	0.23	0.61	0.44	0
16	0.83	0.40	0.62	0 27	0
17	0.92	0.48	0.63	0.19	0
18	0.89	0.59	0.65	0.08	0
19	0.98	0.61	0.69	0.06	0
20	1.06	0.55	0.69	0.12	0

The probable deficiencies should be kept in mind, and the 99%-year storage should be computed if only to indicate the deficiency to be expected during some unusually dry series of years. The storage diagrams indicate the amount of such deficiencies. For example, consider a stream having a $CV = 0.24$ and a 40-day storage constant; the computed storage for 0.90 draft for 90% of the years is 0.53 times the mean annual run-off. About once in 20 years, as indicated on the 95%-year diagram, storage in excess of 0.65 will be needed, and once in 100 years, or 99% of the time, storage exceeding 0.88 will be required. Therefore, if 90%-year regulation is provided there will be a probable deficiency of 0.65—0.53, or 0.12 times the mean annual run-off once in 20 years, and a deficiency of 0.88—0.53, or 0.35 times

the mean annual run-off once in 100 years. These deficiencies may be distributed over more than 1 year, and they may occur not 20 or 100 years hence, but at any time.

Having in mind the thought of caution in the use of computed flows, it may be mentioned that although the storage diagrams are carried through to drafts equal to the entire mean stream flow it was not expected that an engineer would attempt to use the higher flows in actual operations. This extreme development was for purposes of study, and to permit tests of operation at maximum theoretical draft. The diagrams take no account of evaporation. This must be computed separately, and, in some instances, with large annual storage, may reach enormous figures.

Referring to Mr. Craig's objection that the mean law of error does not apply to stream flow as there are more dry years than wet ones—this is provided for by the skew curve. The curve from which the run-off values of the artificial record were drawn, and the other run-off percentage-of-time curves assumed as applicable to streams having $CV = 0.1$ to 0.3, have 46% of their terms above, and 54% below, the average. In other words, there are 8 more dry years than wet ones in a 100-year period. As to dry years succeeding dry years, that tendency is found in many instances in the artificial record, and, in fact, is inevitable because of the preponderance of dry years.

Mr. Armstrong refers to the large possible error in computing CV from short records and asks what length of record is required to obtain a satisfactory value of CV. Mr. Hazen has discussed probable errors at some length.*

Assuming any probable error in CV the error in yield may be found directly from the storage curves for any given storage. For example, if the computed CV is, say, 0.20, with a probable error of 20%, the yield for any storage will lie between the values of the $CV = 0.16$ and the $CV = 0.24$ curves.

Mr. Hall apparently does not entirely understand the use of the term, "monthly" storage. This has no significance in the division of the years by the peaks appearing on the mass curve, but is used to define that portion of the seasonal storage which is due solely to the monthly or daily fluctuations of flow, as distinguished from the storage required because of the annual deficiency of that year. The storage year, taken from peak to peak of the mass curve, is used only to determine CV, and not the monthly storage. It can be taken just as well from the end of one dry season to the end of the next, as Mr. Hall advocates. The result is the same, and the year may be either longer or shorter than 12 months. The point is that the storage year thus taken coincides with the proposed use of the storage, whereas, the calendar year, or other 12-month period, will not always do so.

Mr. Hall further states that one weakness of the writer's method is that with high rates of draft, etc., the maximum depletion may be, in fact, not coincident with the year of minimum run-off. He is mistaken in thinking the writer makes this assumption. It seems to be true, as stated by Mr. Foster, "that the maximum annual storage is required in the same year in

* "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), Table 20, p. 1616.

which seasonal storage is a minimum", but this refers only to years considered singly.

It is quite possible that maximum depletion of storage, in case of high draft, may even occur in a wet year, depending on the CV of the stream and the rate of draft. With a low CV and moderate draft, requiring small annual storage, maximum depletion may occur in a wet year due to a freakish regimen of flow. With a large CV and very high draft, maximum depletion probably will take place in a dry year, but not necessarily the dryest year. In this case the total quantity of annual storage required, plus the monthly storage, will exceed the seasonal storage for any one year, therefore, the excess flow of the wetter part of the year will be stored for use in the dryer part, during the years following partial or total depletion of the annual storage. The crisis will be due usually to shortage of annual storage.

Conditions appear to change gradually as changes in CV and the draft are considered. Thus, for all values of CV, beginning with the lower drafts, the critical condition will be due solely to shortage of seasonal storage, no annual storage being provided and no annual deficiencies expected. In case of a small CV, an increase in draft beyond a certain point will require a comparatively small amount of annual storage, plus an amount of monthly storage that is nearly equal to the computed seasonal storage. The crisis here will be due, probably, as before, to a shortage of water during the dryer part of an average or slightly dry or wet year, of unusual regimen of flow.

For the extreme case of a very large CV and very high draft, the total annual storage will greatly exceed the required monthly storage, and the final failure of the draft will be due to a shortage of annual storage. As an example, computations for the Croton River are given as follows: For 1.0 draft, 95% regulation, the annual storage of 1.85 times the mean annual flow is found to be necessary. The computed total storage for this draft is 2.06, or only 0.21 more than the required annual storage. Probably the largest amount of storage to be used in any one year is about 0.56 (indicated on Fig. 10), and this will be required only in a 95% dry year. With this large storage capacity it is evident that unless the reservoir is already nearly full, there will always be plenty of capacity available to store the excess flow of the flood seasons, for use in the dry seasons. Therefore, no shortage can occur until the annual storage is nearly or quite exhausted.

Mr. Hall feels that the writer's method of plotting annual storage gives undue weight to periods of maximum depletion. As the method corresponds to actual reservoir operation, it seems, however, to be a reasonable one.

Messrs. Hall, Hatch, and Eliassen criticize the method of drawing the 1 000-year sequence of annual flow. The adopted method, using 50 run-off values and keeping out each one as drawn, until the entire 50 were taken, may be open to serious objections, some of which were considered at the time the work was done. It appeared expedient to draw them in this way, however, for several reasons. It was hoped that the 20 drawings of 50 terms each would eliminate the possibility of a grouping of years, such as would not occur in an average period of even 1 000 years, and surely not in an average period

of 50 or 100 years. Mr. Hatch states that "it becomes evident, therefore, that one chance drawing of the 1 000 values as undertaken by the author is not dependable"; but does the method used constitute one chance drawing of 1 000 values? It was thought that the drawings as made would provide a record, perhaps not truly representative of a 1 000-year period but probably useful in determining the storage for an average 100-year period. Perhaps a better method can be devised. As drawn a constant 50-year $C\ V$ was insured, which was considered desirable; and the record could be completed with a reasonable expenditure of labor in a mimimum of time.

Mr. Guy states that the 1 000-year record appears to be based on the assumption that no annual discharges will occur in the future that are not within the range of past short-period records. This is not the case. The 50 annual run-off values were taken from a theoretical curve, the lower part of which is illustrated in Fig. 12 for various values of $C\ V$. This curve, as drawn for a CV of 0.24, is shown on the upper part of Fig. 10, alongside the actual run-off curve of the Croton River. The run-off of the dryest of these 50 years was taken as the average value for the 99% year; that is, the diagram was divided into 50 spaces, each spanning 2% of the years, and the values were read in the middle of each space. The run-off of this fiftieth, or dryest, year is not the lowest to be expected, say, once in 1 000 years, but is low enough, it was thought, to give satisfactory results in the study. It seems to be the case that for high drafts the cumulative deficiencies of a series of dry years, and not a single-year deficiency, are most important.

Mr. Guy also discusses the assumption that annual run-offs may occur in any chance order. This, as he states, may be open to question, but the writer cannot follow him in the thought that if so it would be advisable to take at once the worst possible sequence as the basis for storage requirements. The difficulty found in many instances in ascertaining the true mean, or even an approximation to it, is very real, and not to be overlooked. However, engineers are confronted with the task of estimating the useful and dependable flow, to the best of their ability, and without respect to the length of the available history of the stream. It does not assist greatly in this work to be told that their computed mean may be considerably in error unless the error can be eliminated; nor does it appear of great value to be told, as Mr. Streiff states, that flow records are not fortuitous numbers, but are subject to a number of long and short-term cycles, unless it is explained definitely how these cycles can be brought into the computation. That sunspots have an effect on rainfall, or that rainfall and stream flow may be subject to long and short-term cycles, may be true; but does any one at this time know how to utilize this knowledge in computations of stream flow? The writer has seen data indicating a gradually increasing annual rainfall at one point, and a gradually decreasing annual rainfall at another point, extending over a long period of years. He has seen unusually high run-off given for a stream for a certain year, while the run-off of another stream was unusually low for the same year. He has found that the areas and intensities of rainstorms vary tremendously, and that an almost unending series of computations might

be made on the possible flows resulting from the combined discharge of a number of tributaries. The whole subject is extremely complicated.

There seems no way of interpreting and applying all the laws governing stream flow; but by grouping them in their combined effect, as a modified series of fortuitous numbers, it is possible to extend the known history of a stream's discharge with a large degree of probability. If the existing history is very short and of uncertain value, it is not expected that the extrapolation will be as reliable as if it were based on better data. In such circumstances the engineer will check the run-off, CV, and storage constant by rainfall-run-off computations, comparison with other similar streams, etc.; but after he has learned all that is possible from every source, he can still use the storage-draft curves to advantage.

In closing this discussion the writer suggests to any engineer who may be tempted to use the diagrams blindly, that he read over the part that is so often of greatest interest—that is, the discussions. Then again read the discussion by Mr. Justin.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1642

EXPERIMENTAL DEFORMATION OF A CYLINDRICAL ARCHED DAM*

By B. A. Smith,† M. Am. Soc. C. E.

With Discussion by Messrs. William Cain, Charles W. Comstock, W. H. R. Nimmo, and B. A. Smith.

Synopsis

Some experimental confirmation of the formulas given in the writer's paper on "Arched Dams"‡ seemed to be desirable. With this object in view, a cylinder of India rubber was prepared having removable wooden false bottoms. One of these false bottoms was made conical in form, so that when the cylinder was bound to it, the flare would correspond to zero bending moment at the base; this therefore would represent the case of the base simply supported. The other false bottom was made cylindrical so that when the India rubber cylinder was bound to it, the sides at the base would be vertical, corresponding to the practical case of the base encastrée.

A test specimen of the same kind of India rubber was prepared for the purpose of determining its Young's modulus. A direct determination was also made of this constant using the cylinder itself; the value thus determined differed somewhat from the values obtained from the test specimen.

The actual observations of the deformation lie between the values calculated from Young's modulus as determined from the test specimen and from the cylinder itself. In making the observations measurements were first taken with the cylinder empty, and then filled with mercury. Nothing unusual was observed in the case of the base simply supported, but in the case of the base encastrée, there was an abrupt change of direction at the base. A repetition

* Published in October, 1926, *Proceedings*.
† Civ. and Hydr. Engr., Melbourne, Victoria, Australia.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 2042–2043.

of this experiment, carried out subsequently, in which the displacements near the base were measured inside instead of outside, showed that the shape of the inside surface agreed with the calculated shape.

Determination of the Elastic Constants

A special test piece of the same composition as the cylinder itself was prepared for the purpose of determining the Young's modulus of the India rubber. Its bulk modulus was assumed to be so large that the volume could be treated as constant during the measurements.

The test piece was 6.3 in. (16.0 cm.) long, 0.51 in. (1.30 cm.) wide, and 0.49 in. (1.25 cm.) thick. Tested by direct stretching, a length of 5 in. elongated as follows:

Load, in pounds.	Length, in inches.
0	5.00
2	5.28
4	5.65

The corresponding values of E_0 (Young's modulus) are 151 and 139 lb. per sq. in., giving a mean of 145 lb. per sq. in., or 10.2 kg. per sq. cm. The specimen was also tested as a beam resting on rollers 6 in. apart. The corresponding value of E was determined by the ordinary deflection formula,

$$D = \frac{W\, l^3}{48\, E\, I}$$

Measurements were made as given in Table 1. In the first pair, the side 0.49 in. long was vertical, and in the second pair the side 0.51 in. long was vertical; the specimen was inverted between each pair of measurements, ample time being given to allow the India rubber to recover from the previous deflection. The modulus in bending is called E_1; its mean value is 174 lb. per sq. in., or 12.2 kg. per sq. cm.

TABLE 1.—Bending of India Rubber Prism.

Load.		Deflection, in inches.	E_1, in pounds per square inch.
In grammes.	In pounds.		
50	0.11	0.55	180
50	0.11	0.60	165
50	0.11	0.51	180
50	0.11	0.54	170

The difference between these results was surprising, but a repetition of the experiment gave similar results, namely, $E_0 = 125$ lb. per sq. in. and $E_1 = 172$ lb. per sq. in.

In case the value of Young's modulus for the cylinder might differ from that in the specimen a separate determination of its value was made. For this purpose the cylinder (with the wooden base removed so that it was open at

both ends) was allowed to deform under its own weight by placing it horizontally on a table in four different positions, namely, with each of four quadrants A, B, C, and D, in turn, on top. The mean elongation, H, of the horizontal diameter in each of the four positions was observed and also the mean contraction, V, of the vertical diameter.

It can be shown that:

$$H = \frac{w\,a^4}{E\,I}\left(2 - \frac{\pi}{2}\right) = 0.4292\,\frac{w\,a^4}{E\,I}$$

$$V = \frac{w\,a^4}{E\,I}\left(\frac{\pi^2}{4} - 2\right) = 0.4674\,\frac{w\,a^4}{E\,I}$$

in which,

a = mean radius of the cylinder = 7.45 cm.

l = length = 18 cm.

$I = \frac{l\,t^3}{12}$ = 3.45 cm.^{4}

t = thickness = 1.32 cm.

w = weight per unit length of the mean circumference.

Since the total weight, W = 1 091 grammes,

$$w = \frac{W}{2\,\pi\,a} = \frac{1\,091}{2\,\pi \times 7.45} = 23.3 \text{ grammes per cm.}$$

After the deformation the mean horizontal diameter was found to be 15.56 cm. and the mean vertical diameter, 14.19 cm.

Hence,

$$H = 15.56 - 14.90 = 0.66 \text{ cm.}$$

$$V = 14.90 - 14.19 = 0.71 \text{ cm.}$$

Using these values it is found that E_1 = 13.5 and 13.7 kg. per sq. cm., respectively. The mean value is E_1 = 13.6 kg. per sq. cm.

No satisfactory method of determining E_0 for the circumferential stress suggested itself, therefore the assumption was made that $E_0 = E_1$ = 13.6 kg. per sq. cm.

Description of Experiments

In making an experiment a set of measurements was taken first with the cylinder empty and then filled with mercury.

Two diameters, $A\,C$ and $B\,D$, at right angles, were measured at heights, 0, 1, 2, 3, * * * 6 in. A, B, C, D thus divide the cylinder into four quadrants. No record was taken of the temperatures. The temperature in the shade recorded at the Weather Bureau (near the University of Melbourne) on the date of Experiment No. 1 (October 13, 1924) was 76° Fahr.; on the date of Experiment No. 2 (November 11, 1924), it was 65° Fahr.; and on the date of the repetition of Experiment No. 2 (December 12, 1925), 69° Fahr.

The calipers used (both inside and outside types) were fitted with clamping screw and slow-motion attachment. The measurements were estimated to 0.01 cm., but the error of any reading possibly amounted to 0.03 cm. on

account of the difficulty of ensuring that the calipers did not press into the rubber.

In making the outside measurements, one observer would hold a leg of the calipers in the position, say, A_3 (Position A, 3 in. above the base), while the other observer set the other leg so as to be just in contact at C_3 (Position C, 3 in. above the base); the spread of the legs, in centimeters, was then read.

The results of the measurements for the base simply supported (taken on October 13, 1924) are shown in Fig. 1 (*a*). For the case of the base encastrée the results of the measurements (taken on November 12, 1924) are shown in Fig. 1 (*b*). In the latter experiment there was an abrupt change of direction in the surface at the base when the cylinder was full of mercury; as nearly as could be judged by the eye (using a straight-edge) it was straight from 0 to 1 in. above the base.

The abrupt change of direction was due to the considerable local stresses at the junction of the binding wire and the free surface of the India rubber. The same phenomenon can be observed by placing a piece of India rubber $\frac{1}{2}$ in. square and 3 or 4 in. long in a vise and bending it over by one finger held about 1 in. from the jaws. The side away from the finger remains practically straight and shows an abrupt angle at the vise while the side on which the finger is pressed is curved.

A repetition of this experiment was made on December 12, 1925, the internal diameter, $A\,C$, being measured at each $\frac{1}{8}$ in. of height from 0 to 1 in. These measurements are also recorded in Fig. 1 (*b*). The displacements are taken as one-half the difference in diameter at the base and at the given height.

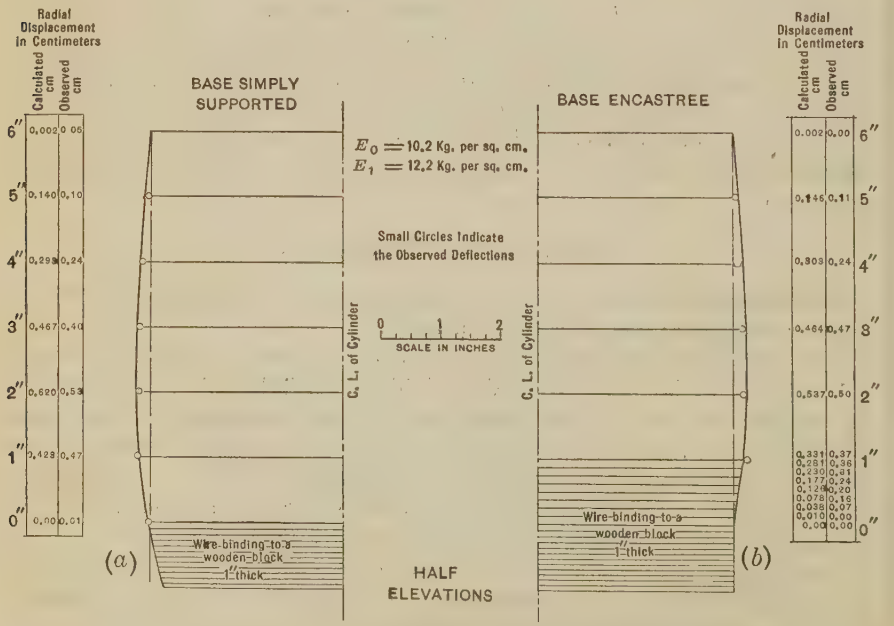

FIG. 1.—EXPERIMENTS ON DEFLECTIONS OF AN INDIA RUBBER CYLINDER FILLED WITH MERCURY.

Calculation of Displacements

1.—Base Simply Supported.—The pressure at the depth, z, is,

$$p = w\,(d - z) = w\,d\left(1 - \frac{z}{d}\right)$$

The limiting conditions are: (1) when $z = 0$, $u = M = 0$, that is, $u = \frac{d^2\,u}{d\,z^2} = 0$, and (2) when $z = d$, $M = F = 0$, that is, $\frac{d^2\,u}{d\,z^2} = \frac{d^3\,u}{d\,z^3} = 0$.

Using these conditions in Equations (16) and (17) of the writer's paper on "Arched Dams",* it is found that, neglecting terms involving $e^{-\frac{\mu\,d}{\sqrt{2}}}$, the constants,

$$A = B = D = 0$$

and the constant,

$$C = -\frac{w\,d\,R^2}{E_0\,t}$$

and, therefore,

$$u = \frac{w\,d\,R^2}{E_0\,t}\left(\left(1 - \frac{z}{d}\right) - e^{-\frac{\mu\,z}{\sqrt{2}}}\cos\frac{\mu\,z}{\sqrt{2}}\right)$$

in which,

w = weight of 1 cu. cm. of mercury = 13.6 grammes per cu. cm.
d = depth, in centimeters = $6 \times 2.54 = 15.24$ cm.
R = mean radius = 7.45 cm.
t = thickness = 1.32 cm.
E_0 = Young's modulus { = 10.2 kg. per sq. cm. from test specimen. = 13.6 " " " " " cylinder.
E_1 = Young's modulus for bending { = 12.2 kg. per sq. cm. from test specimen. = 13.6 kg. per sq. cm. from cylinder.

$$\mu^4 = 12 \times \frac{E_0}{E_1} \times \frac{1}{t^2\,R^2} = \frac{12\,E_0}{E_1\,t^2\,R^2}$$

z = height above base.
e = base of Naperian logarithms.
u = displacement measured along radius, outward.

Table 2 shows the numerical values of constants required for calculation of the displacements.

TABLE 2.—Computed Numerical Values.

Values of E_0, in kilogrammes per square centimeter.	Values of E_1, in kilogrammes per square centimeter.	Numerical values of $\frac{\mu}{\sqrt{2}}$.	Numerical values of $\left(1 - \frac{\sqrt{2}}{\mu\,d}\right)$.	Numerical values of $\frac{w\,d\,R^2}{E_0\,t}$.
10.2	12.2	0.4013	0.8365	0.8544
13.6	13.6	0.4197	0.8437	0.6408

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 2042–2043.

Table 3 gives the displacement at each inch corresponding to the values of E_0 and E_1 and, for comparison, the observed displacements from Fig. 1 (*a*).

TABLE 3.—Displacements for Base Simply Supported (See Fig. 1 (*a*)).

z, in inches.	Computed Values of u, in Centimeters.		Observed values of u, in centimeters.
	$E_0 = 10.2$ $E_1 = 12.2$	$E_0 = 13.6$ $E_1 = 13.6$	
0	0.000	0.000	— 0.01
1	0.423	0.427	0.47
2	0.620	0.468	0.53
3	0.467	0.347	0.40
4	0.293	0.218	0.24
5	0.140	0.104	0.10
6	0.002	0.001	— 0.05

2.—Base Encastree.—As before, $p = w\,d\left(1 - \frac{z}{d}\right)$. The limiting conditions are:

When $z = 0$,

$$u = \frac{d\,u}{d\,z} = 0$$

and when $z = d$,

$$\frac{d^2\,u}{d\,z^2} = \frac{d^3\,u}{d\,z^3} = 0$$

TABLE 4.—Displacements for Base Encastrée (See Fig. 1(*b*)).

z, in inches.	Computed Values of u, in Centimeters.		Observed values of u, in centimeters.
	$E_0 = 10.2$ $E_1 = 12.2$	$E_0 = 13.6$ $E_1 = 13.6$	
0	0.000	0.000	0.00
1/8	0.010	0.008	0.00
1/4	0.038	0.029	0.07
3/8	0.078	0.058	0.16
1/2	0.126	0.095	0.20
5/8	0.177	0.134	0.24
3/4	0.230	0.172	0.31
7/8	0.281	0.210	0.36
1	0.331	0.265	0.37
2	0.537	0.413	0.50
3	0.464	0.348	0.47
4	0.303	0.224	0.24
5	0.146	0.106	0.11
6	—0.002	—0.001	0.00

from which it follows that, neglecting $e^{-\frac{\mu\,d}{\sqrt{2}}}$,

$$A = B = 0$$

$$C = -\frac{w\,d\,R^2}{E_0\,t}$$

$$D = -\frac{w\,d\,R^2}{E_0\,t} \times \left(1 - \frac{\sqrt{2}}{\mu\,d}\right)$$

so that,

$$u = \frac{w\,d\,R^2}{E_0\,t} \times \left[\left(1 - \frac{z}{d}\right) - e^{-\frac{\mu z}{\sqrt{2}}} \left\{ \cos \frac{\mu z}{\sqrt{2}} + \left(1 - \frac{\sqrt{2}}{\mu d}\right) \sin \frac{\mu z}{\sqrt{2}} \right\} \right]$$

The values of the constants are the same as for the base simply supported.

Table 4 gives the calculated displacements for the different observed values of E and the actual measured displacements.

Acknowledgments

The writer's thanks are due to Professor J. N. Greenwood and the Staff of the Engineering School of Melbourne University for their valuable assistance in the preparation of apparatus and conduct of experiments; also to his Assistant, Mr. C. L. Sanders, who carried out the actual measurements with him.

DISCUSSION

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The notable solution of "Arched Dams" given by the author† is the most complete, for this arched dam, that has yet appeared, in that the shear between supposed horizontal arches was included in the analysis. Where the dam is fixed (encastrée) at the base, the tentative method‡ presented by F. A. Noetzli, M. Am. Soc. C. E., which ignores this shear, affords a good solution. This tentative method also applies to very thick dams if correct formulas for deflection are used.§

However, when the dam is simply supported at the base the tentative method fails, since the line of deflection of a supposed vertical cantilever makes an unknown angle with the vertical. Mr. Smith's method affords a solution. It applies strictly to a dam in a comparatively level valley, and is, perhaps, only roughly approximate when the dam is situated in a deep V-shaped canyon.

In all theories pertaining to segmental arched dams it is assumed that the normal load on the supposed horizontal arches is uniformly distributed. This may not be correct; but it is doubtless a sufficiently close approximation, as thin circular dams have not, hitherto, failed, as they probably would if the normal load were not approximately uniform.

For the cylindrical tank on a level base, the conditions postulated by the author are fulfilled; since, at the same level, they are the same throughout the full circumference. Hence, it was to be expected that the deflections for a rubber tank filled with mercury should agree fairly well with computed stresses, which was found to be the case for the base either fixed or simply supported. The author has done a real service in making these experiments which tend to verify the theory.

The writer was particularly interested in the case of the tank simply supported on a conical base the elements of which, as measured on Fig. 1 (*a*), make angles of about 14° with the vertical. This value seemed so large that it was checked. By differentiating the value of u and placing $z = 0$ in the result,

$$\tan \phi = \frac{d\,x}{d\,z}\Big)_{z\,=\,0} = \frac{w\,d\,R^2}{E_0\,t}\left(\frac{\mu}{\sqrt{2}} - \frac{1}{d}\right)$$

in which, ϕ = the angle that the deflection curve makes with the vertical. Using the numerical values given in Table 2:

$$E_0 = 10.2;\ E_1 = 12.2;\ \tan \phi = 0.286;\ \phi = 16°.$$
$$E_0 = E_1 = 13.6;\ \tan \phi = 0.227;\ \phi = 12°\ 47'.$$

The average is 14° 23′, which shows that the angle used is practically correct.

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† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

‡ "Gravity and Arch Action in Curved Dams" *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

§ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 475–547, "Stresses in Thick Arches of Dams," by B. F. Jakobsen, M. Am. Soc. C. E., with discussion and a solution by the writer by the work method.

The wire that binds the rubber cylinder around the conical surface evidently causes shear in the rubber, since each horizontal slice of infinitesimal height slips over one just below.

The case is much more complicated for an arched dam, where the foundation may yield appreciably. For simplicity, consider the foundation rigid for the case of the Wooling Dam simply supported at the base. The solution was affected by the writer by aid of Mr. Smith's formulas and the use of Michell's functions. The values of u, M, and T are given in Table 1* and are shown graphically in Figs. 6 and 7† of the writer's discussion of Mr. Noetzli's paper. The deflection curve for the water load makes the angle at the base with the vertical about equal to 0° 4′ 48″, or, in radians, $\phi = 0.00140$. To compute the modulus in shear (G) for $E = 2\,000\,000$ lb. per sq. in., assume Poisson's ratio $= \frac{1}{3}$, or $m = 5$, and use the known formula,

$$\frac{E}{G} = \frac{2m + 1}{m}$$

Then,

$$G = \frac{5}{12} E = \frac{1}{6} (5\,000\,000) \text{ lb. per sq. in.}$$

The unit shear near the base, corresponding to the displacement, ϕ, is,

$$\phi G = 0.0014 \times \frac{5\,000\,000}{6} = 1\,166 \text{ lb. per sq. in.}$$

This value is doubtless incorrect, since practically nothing is known of the value of m for thick beams; besides, the voussoir considered is not in pure shear, being acted on by arch thrusts, by loads due to water and the weight of dam, and by couples giving the moments, M, on a supposed vertical cantilever.

Since the shear between the supposed horizontal arches, per unit length of dam, is given by Mr. Smith's Formula (2)‡, $F = -\frac{d M}{d z}$, the best solution would appear to be that based on this formula.

The writer believes that the angle, ϕ, at the base is caused by shear and has given the following reason for it:* Bearing in mind that the deflection, u, and the arch thrust, T, are both zero at the base, and since T (compression) increases rapidly above the base, the consequent shortening of a horizontal arch near the base gives a displacement of its crown down stream, the displacement increasing with the height above the base. It is plainly a case of shear; each horizontal arch of infinitesimal height slipping over the one just below it. As this subject is not well understood, the author's views relative to it will be appreciated.

Charles W. Comstock,§ M. Am. Soc. C. E. (by letter).—Discussion of this paper seems to be inseparable from that of the paper entitled "Arched Dams"‖ by the same author.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 312.
† *Loc. cit.*, p. 313.
‡ *Loc. cit.*, Vol. LXXXIII (1919–20), p. 2040.
§ New York, N. Y.
‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

The purpose of the present paper is to compare observed deformations of an India rubber model with corresponding values computed by the process set forth in the earlier paper. These comparisons are summarized in Table 3 ("for base simply supported") and Table 4 ("for base encastrée").

Table 5 is a copy of Table 3 with the addition of a column of differences between observed and computed values and of a column of percentage excesses of observed above computed values. Likewise Table 6 is a copy of Table 4 with similar additions.

TABLE 5.—Comparison of Observed and Computed Deformations, Base Simply Supported.

z, in inches. (1)	Computed values of u, in centimeters. (2)	Observed values of u, in centimeters. (3)	Difference, in centimeters. (4)	Percentage excess of observed above computed values. (5)
0	0.000	−0.01	−0.010	.....
1	0.427	+0.47	+0.043	+10.0
2	0.468	+0.53	+0.062	+13.3
3	0.347	+0.40	+0.053	+15.2
4	0.218	+0.24	+0.022	+10.0
5	0.104	+0.10	−0.004	− 4.0
6	0.001	−0.05	−0.051	.....

The author states, "the error of any reading possibly amounted to 0.03 cm." If it be assumed that the measured deformations are in excess of the actual by this amount the percentages in Table 6 would be 38, 124, 79, 57, 63, 57, and 28, instead of 141, 176, 110, 79, 80, 71, and 40, as given in Column (5).

TABLE 6.—Comparison of Observed and Computed Deformations, Base Encastrée.

z, in inches. (1)	Computed values of u, in centimeters. (2)	Observed values of u, in centimeters. (3)	Difference, in centimeters. (4)	Percentage excess of observed above computed values. (5)
0	0.000	0.00	0.000	.....
1/8	0.008	0.00	−0.008	−100
1/4	0.029	0.07	+0.041	+141
3/8	0.058	0.16	+0.102	+176
1/2	0.095	0.20	+0.105	+110
5/8	0.134	0.24	+0.106	+ 79
3/4	0.172	0.31	+0.138	+ 80
7/8	0.210	0.36	+0.150	+ 71
1	0.265	0.37	+0.105	+ 40
2	0.413	0.50	+0.087	+ 21
3	0.348	0.47	+0.122	+ 35
4	0.224	0.24	+0.016	+ 7
5	0.106	0.11	+0.004	+ 4
6	−0.001	0.00	−0.001	−100

Even with this favorable assumption as to errors of observation, it can hardly be considered that there is substantial agreement between computed and

measured distortions. If the percentage differences were nearly constant throughout, it might properly be assumed that the modulus of elasticity of the material of the model differed from the value used in the computations. However, this is not the case, and the only alternative would seem to be that the stress distributions assumed in the earlier paper, either as to the arch, or the cantilever, or both, are not the actual distributions.

The author's analytical method consists in equating the radial distortion of a ring to the deflection of a cantilever perpendicular to the plane of the ring. The result is a differential equation of fourth order and first degree, which is integrated in terms of exponential and trigonometric functions for a cylinder of constant thickness; and in terms of hyper-geometric series for a cylinder with a thickness that varies linearly with the height.

The method is not new, having been used by Silas H. Woodard, M. Am. Soc. C. E., in the design of the Lake Cheesman Dam.* Mr. Woodard, however, contented himself with equating deflections at five or six points of the height, making no attempt to derive a general formula for deflection at any point. The principle involved is probably sound provided the assumed stress distributions for arch and cantilever are correct.

In his previous paper the author assumed† for the arch "that the bending moment vanishes throughout and that the segment is subjected to a uniform compressive stress (arch thrust) throughout its length so that the elastic compression * * * is constant throughout." This is clearly not the case. Even for the cylindrical model used in his tests, with a thickness of 17.7% of the mean radius, the circumferential stress was 18% greater at the inside surface than at the outside, although it is true that there was no bending moment about a line parallel to the axis of the cylinder. Had the model comprised only a segment of the circumference, instead of a complete cylinder, the assumption of no bending moment in the arch would have been obviously incorrect and the use of deflections based on such assumption entirely unjustified.

For a vertical cantilever, the author makes use of the common theory of beams. With such proportions as existed in his model this seems entirely reasonable, but for the sections of curved dams like the Cheesman, Roosevelt, and Arrowrock, or even the Shoshone Dam, there is no basis for assuming linear distribution of stress on horizontal planes; and deflections calculated by the common theory are almost certainly seriously in error.

If, as the writer believes, the author's formulas for deflections of both arches and cantilevers are incorrect, the remainder of his arch dam theory becomes simply an interesting mathematical exercise with no relation to the facts and therefore without practical usefulness. His own tests seem to demonstrate this.

The author has begun in the middle of the problem. One necessary preliminary to a proper application of his analysis is an adequate theory of the thick elastic arch. This has not yet appeared. For a structure of such proportions as the Stevenson Creek Test Dam, the ordinary theory of curved

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), pp. 108 *et seq.*

† *Loc. cit.*, Vol. LXXXIII (1919–20), p. 2030.

beams is probably sufficient, but for more usual proportions a more scientific analysis is needed. If the Stevenson Creek Dam had the proportions of the author's model it would be about 17 ft. thick. It is doubtful whether any one would feel justified in assuming uniform circumferential stress and no bending moment in such a structure.

Another preliminary problem, and one even more difficult of solution, is that of stress distribution in a beam the cross-section of which is rapidly varying and of the same order of magnitude as the length of the beam. When these problems shall have been successfully solved the author's method may be useful. Until that time it does not seem to have any place in the design of arch dams.

W. H. R. Nimmo,* Assoc. M. Am. Soc. C. E. (by letter).—The experiments described in this paper generally confirm the analysis of the problem given by Mr. Smith in his previous paper† but there is considerable variation between the several experimental values found for E_0 and E_1. The values of E_0 (Young's modulus) determined from the direct tensile tests should be 143 and 123 lb. per sq. in., giving a mean of 133 lb. per sq. in., or 9.35 kg. per sq. cm. The values of 151 and 139 lb. per sq. in. given in the paper are presumably in error and do not agree with the value of 125 lb. per sq. in. obtained from a repetition of the experiments.

The formulas given by the author for H and V may be derived from the theory of thin arches, as usually developed for pipes and sewers, and involve the assumptions that the axial length of the ring is short and that the thickness, t, is so small in comparison with the radius that the effect of curvature on the moment of resistance of the beam may be neglected. On these assumptions it can also be shown that there are: A circumferential thrust at the crown, $P_T = -\frac{w\,a}{2}$; a circumferential thrust at the ends of the horizontal diameter, $P_S = \frac{\pi\,w\,a}{2}$; a circumferential thrust at the bottom, $P_B = \frac{w\,a}{2}$; a circumferential moment at the crown, $M_T = \frac{w\,a^2}{2}$; a circumferential moment at the ends of the horizontal diameter, $M_S = -\,0.5708\,w\,a^2$; and, a circumferential moment at the bottom, $M_B = 1.50\,w\,a^2$; in which, a positive value of P denotes compression and a positive moment, M, is one that produces tension at the inner surface of the ring. In the case of a ring of unit axial length, the extreme fiber stress at the inner surface at the crown due to the moment, M_T, will be,

$$f_T = -\frac{6\,M_T}{t^2}$$

(the negative sign denoting tension) and the corresponding unit deformation will be,

$$\frac{f_T}{E_0} = -\frac{6\,M_T}{E_0\,t^2}$$

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† "Arched Dams," Transactions, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 2042.

Owing to the effect of Poisson's ratio, the circumferential moments will give rise to axial deformations of varying sign at different parts of the ring, the surface of which will, therefore, undergo considerable warping. In the case of a long cylinder, however, such warping could not take place to any great extent and the axial deformations would be largely neutralized by axial moments, the magnitude of which would be, approximately, m times the circumferential moment at the corresponding part of the cylinder (m denoting Poisson's ratio). The axial moment at the crown would give rise in its turn to a unit circumferential deformation of the extreme fiber at the inner surface of $\frac{6\,m^2\,M_T}{E_0\,t^2}$ and, therefore, the net unit deformation of the extreme fiber at the crown at the inner surface would be,

$$\frac{6\,m^2\,M_T}{E_0\,t^2} - \frac{6\,M_T}{E_0\,t^2} = \frac{6\,M_T}{t^2}\left(\frac{m^2-1}{E_0}\right)$$

Since the extreme fiber stress at the inner surface at the crown is $-\frac{6\,M_T}{t^2}$, the modulus of elasticity for the circumferential stresses in the cylinder is,

$$E_c = -\frac{6\,M_T}{t^2} \div \frac{6\,M_T}{t^2}\left(\frac{m^2-1}{E_0}\right) = \frac{E_0}{1-m^2}$$

For rubber, the value of m is about $\frac{1}{2}$ and, therefore, $E_c = \frac{4}{3}\,E_0$. If $E_0 = 9.35$ kg. per sq. cm., then the value of E_c becomes 12.5 kg. per sq. cm.

The experimental value of $E_1 = 13.6$ kg. per sq. cm. is about 9% in excess of the value deduced for E_c and the difference may be due partly to an error in the adopted value of $m = \frac{1}{2}$. It may also be largely due to the effect of curved beam action, which is neglected in deducing the formulas for H and V.

TABLE 7.—Values of Radial Displacements, u, in Centimeters.
(E = 177 lb. per sq. in. = 12.5 kg. per sq. cm.)

z, in inches.	Base Simply Supported.		Base Encastree.	
	Computed.	Observed.	Computed.	Observed.
0	0.000	−0.01	0.000	0.00
1/8	.....	.....	0.009	0.00
1/4	.....	.....	0.032	0.07
3/8	.....	.....	0.068	0.16
1/2	.....	.....	0.103	0.20
5/8	.....	.....	0.146	0.24
3/4	.....	.....	0.187	0.31
7/8	.....	.....	0.239	0.36
1	0.465	0.47	0.278	0.37
2	0.509	0.53	0.450	0.50
3	0.378	0.40	0.379	0.47
4	0.288	0.24	0.244	0.24
5	0.113	0.10	0.115	0.11
6	0.001	−0.95	−0.001	0.00

If the value of $E_c = 12.5$ kg. per sq. cm. be adopted for both vertical and horizontal stresses in the cylinder when the latter is filled with mercury, then the values for u given in Table 7 are obtained. It will be noted that, for the case of the base simply supported, there is a closer agreement between the computed and observed values of u in Table 7 than obtains in Table 3. For the case of the base encastrée, the results, for values of z exceeding 3, are better than those in Table 4; but for the lower part of the cylinder the results are not so good. In the latter case, since there is a displacement of more than $\frac{1}{8}$ in. at a height of 1 in. above the base, considerable curvature exists in vertical radial planes and, therefore, any formulas that neglect the effect of curvature will not give correct results.

In the beam experiments described in the paper, the deflection amounts to more than one-twelfth of the span. Consequently, the value of $E_1 = 172$ lb. per sq. in., deduced from the ordinary straight beam formula, $D = \frac{W l^3}{48 E I}$, is not comparable with the value of E_0 obtained from the direct tensile tests.

B. A. Smith,* M. Am. Soc. C. E. (by letter).—Professor Cain, in his kind criticism, directs attention to the shear near the base of an arched dam. The shear or traction, F, per unit length, which is exerted by the material above a given horizontal section or that below it, depends on the rate of variation of the bending moment, M, per unit length, about the horizontal tangent at the given point as is expressed by the equation,†

$$\frac{d\,M}{d\,z} + F = 0$$

The effect of the ring tension (or thrust), T, per unit height, on the bending moment is negligible compared with that of the shear, F. At any height, z, above the base, the relation between the shear, F, the ring tension, T, and the water pressure, p, is given by Equation (1) of the writer's paper on "Arched Dams",† namely,

$$\frac{d\,F}{d\,z} - \frac{T}{R} + p = 0$$

At the base the ring tension or thrust vanishes so that it can have no effect on the shear which is given by‡ $F = \frac{w\,d}{\mu\sqrt{2}}$, if the base is simply supported or, $F = \frac{w\,d}{\mu\sqrt{2}}\left(2 - \frac{\sqrt{2}}{\mu\,d}\right)$, if the base is encastrée, the constants having the values defined in the paper (page 709), namely,

$w = 13.6$ grammes per cu. cm.
$d = 15.24$ cm.
$u = 0.5674$ to 0.5934.

* Civ. and Hydr. Engr., Melbourne, Victoria, Australia.
† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2040, Equation (2).
‡ *Loc. cit.*, p. 2040, Equations (2) and (3) and p. 2043, Equation (17).

Using these values the shear is found to be, in the case of the India rubber model, for the base simply supported: $F = 250$ to 260 grammes per cm., and for the base encastrée, $F = 450$ to 470 grammes per cm.

At a small height above the base, $T = p\,R$, and F becomes constant; and, since F is zero at the surface, it vanishes from above the point where $T = p\,r$.

The writer's thanks are due Professor Cain for pointing this out, to which, perhaps, too little attention was given in the earlier paper, although due account was taken of it in the course of the work.

Mr. Comstock criticizes the work from two standpoints:

(1) That the discrepancies between the calculated and observed deflections (displacements) amount to considerably more than 100 per cent.

(2) That the fundamental assumptions are erroneous.

As Criticism (2), if just, is fatal, it will be considered first.

Mr. Comstock's statement that "the author's analytical method consists in equating the radial distortion of a ring to the deflection of a cantilever perpendicular to the plane of the ring", is incorrect.

Consider for simplicity, the case of a complete cylindrical tank of uniform thickness filled with liquid (the cylinder is supposed to be thin). The equations of equilibrium of an element of the cylinder under the forces and couples acting on it are written down and the following assumptions are then made:

(*a*) That the bending moment about a vertical generator is negligible.*

(*b*) That the ring tension is proportional to the radial displacement outward.

(*c*) That the bending moment about a horizontal tangent at any point is proportional to the curvature of the vertical generator at that point.

If these three assumptions are granted (the second and third being merely restatements of Hooke's law), the solution obtained follows as mere mathematical deduction.

It must be remembered that the cylinder is supposed to be thin and, as Mr. Comstock justly points out, a cylinder the thickness of which is about one-sixth its radius can hardly be called thin. Minute accordance between the calculations and the measurements, therefore, is not to be expected.

Mr. Comstock states that "the principle involved is probably sound provided the assumed stress distributions for arch and cantilever are correct". No such assumption is made; the assumptions made are the three just given. The ring thrust (or tension) and bending moment above a horizontal tangent are calculated from the displacement, u, which is determined quite generally from the equations of equilibrium of any element of the cylinder. The writer does not suggest that the work applies to thick arches, but he has applied the work to many cylindrical tanks and arched dams with satisfactory results.

As to the first criticism, the large percentage errors occur, as might be expected, with the small displacements; for the larger displacements, the errors are of the same order as the discrepancies in the values of Young's modulus determined by different experiments (10.2 to 13.6 kg. per sq. cm.).

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2029, and Equation (3), p. 2040.

The writer agrees with Mr. Comstock that no satisfactory solutions have yet appeared for the stresses in a thick arch, or for the stresses in a beam of rapidly varying section; but suggests that until such solutions do appear a tentative solution for a thin arch is better than nothing.

Mr. Nimmo points out quite correctly that if the value of Young's modulus for simple tension is E_0, the corresponding value for stresses in a cylinder will be, $E_1 = \frac{E_0}{1 - m^2}$, m being Poisson's ratio for the material. For concrete, m is about $\frac{1}{8}$ so that, nearly enough, $E_1 = E_0$; but for India rubber, $m = \frac{1}{2}$ so that $E_1 = \frac{4}{3} E_0$, as was found experimentally.

The reason for the discrepancy between the values of E determined from the beam and from the direct tension experiment is possibly the fact that India rubber does not obey Hooke's law for large deformations. It is known that for even moderately large deformations the volume of the test piece remains constant, that is, in effect, Poisson's ratio is $\frac{1}{2}$.

If, therefore, l_0 is the original length, and A_0, the original area of a specimen tested in simple tension, and l and A, the length and area after strain,

$$l A = l_0 A_0$$

If W is the added weight, the stress is,

$$P = \frac{W}{A} = \frac{W l}{A_0 l_0}$$

while the strain is,

$$e = \frac{l - l_0}{l_0}$$

Hooke's law is $P = E \times e$, in which E is constant (Young's modulus); that is,

$$\frac{W l}{A_0 l_0} = E \frac{l - l_0}{l_0}$$

or,

$$E = \frac{W l}{A_0 (l - l_0)}$$

In the first experiment, $W = 2$ lb., $l_0 = 5$ in., $l = 5.28$, and $A_0 = 0.51 \times 0.48 = \frac{1}{4}$ sq. in.; hence,

$$E = \frac{2 \times 5.28}{\frac{1}{4} \times 0.28} = 151 \text{ lb. per sq. in.}$$

In the second experiment, $W = 4$ lb., $l_0 = 5$ in., and $l = 5.65$ in.; hence,

$$E = \frac{4 \times 5.65}{\frac{1}{4} \times 0.65} = 139 \text{ lb. per sq. in.}$$

That is, Hooke's law does not hold accurately, and discrepancies between theory, based on the assumption that the law holds, and actual practice must be expected.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1643

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED*

By John Nolen,† M. Am. Soc. C. E.

With Discussion by Messrs. John W. Alvord, William T. Lyle, Thomas Adams, Edward M. Bassett, J. E. Willoughby, Noulan Cauchon, and John Nolen.

There is much in the speaker's mind, and in the minds of most other students of town planning, that is not yet altogether clear. The art and science of town planning still contain many unknown elements. One thing, however, is fairly clear, namely, that town planning, if it is to accomplish its purposes in a worthy manner, must be broad in the professions it employs. If permanent results of a high order are to follow, the engineer, the architect, the landscape architect, and also the lawyer, and those of other allied professions, must be united in a joint enterprise, one which should have variety, harmony, and unity. The planner of the town himself may be an engineer, an architect, or a landscape architect. Professionally, he ought to be technically educated, well trained, and widely experienced in at least one of these three fields, but he must know something vital of them all. Successful town planning cannot be the work of a narrow specialist of a single profession. It calls for versatility. It is engineering plus something; architecture plus something; or landscape architecture plus something; and that "plus" is as indispensable as the direct professional equipment in the more usual and better recognized fields. Moreover, the town planner needs to know both design and construction in those public works and improvements that draw upon and employ the skill of several of these professions jointly, and especially those works that

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

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have constantly the double requirement, if it can be so called, of use and beauty. As some one has said, harmony and beauty must be created on a foundation of the practical. One thing more—the town planner must have the social welfare point of view, for the motive back of all town planning is to improve daily living and working conditions. In this field more than in many others, personality plays a large part.

The general town plan, or "master plan", as it is sometimes styled, is not enough. There must be site or local planning. The details of a town development, of all its various features, must be properly done in both plan and elevation. The third dimension enters vitally into any real town development. The general town plan must have its own merit, its own justification; but as the plan of a building is more fundamental than its elevations, so the town plan itself is more important than any or all of its structures. Nevertheless, convenience, order, beauty, and appropriateness are finally realized only in these structures of the engineers, the architects, and the landscape architects. Mark this point of supreme importance, however, that unless this principle of unity and fitness is followed, these various works of detail may be satisfactory in themselves separately, but as an example of the art of town planning, or as a complete fulfillment of their practical purposes, they will fail. Each separate part must be tested for merit both by itself and by its contribution to the whole.

To understand this subject, some of the chief characteristics of town planning and its point of view must be enumerated, and the nature of engineering, architecture, and landscape architecture indicated briefly, especially as they are employed in community development. Mariemont, Ohio, is a good example of a community developed under the joint effort of men representing various professions. It illustrates problems in design, construction, and organization.

1.—To begin, town planning is a comprehensive, inclusive, synthetic art. Its success depends on having a broad view with a keen sense of proportion, of fitness, and of social values. That is the keynote of the town-planning movement.

2.—Town planning is broad from the point of view of the territory included in its schemes. It embraces wide areas. Dealing only with the parts of a town or with local sections or neighborhoods it cannot work successfully. More and more with the extending radius of modern life, it is becoming regional in character. If well done, town planning design takes its cue from this broad regional viewpoint in which there is a skillful arrangement of each part of a wide territory, assigning it to its most appropriate use and development.

3.—Town planning is comprehensive in embracing all the physical elements of a community. It includes the location, character, and appearance of thoroughfares, railroads, parks and playfields, schools, public and semi-public buildings, street structures, and the sanitary disposal of wastes, etc.

4.—Town planning is broad planning from the viewpoint of time. It is historical. It looks forward, it looks backward. To plan for to-day and to-day alone, or to plan for to-day without regard to yesterday, is not town planning in its full sense. The relationship runs backward and forward, and towns

have a long, long life. They endure for centuries. The human limitation of three-score years and ten has no application.

5.—Broad also is the viewpoint of town planning in that it embraces all sides of man's life—animal, social, intellectual, and spiritual. The most commonplace needs of man as an animal must be properly provided for by town planning; indeed these needs—food, shelter, and a place to work—must be considered before anything else. Man, however, is not merely an animal. Man is a spirit. He aims not only to live, but to live well, with increasing freedom and happiness. That means, if it means anything, planning a town and an environment not only for labor but for leisure; and leisure at its best is not idleness, but a different and a higher form of occupation, the facilities for which must be provided largely by the public. Some of the more worthy and satisfying objects of this steadily increasing leisure, combined as it is with the increase of wealth, are to be found in art and science, in play and sport, in the contemplation of the universe, in the enjoyment of Nature, in friendship and social life. One can readily see the close relation of these objects, if they are to be shared by all, to a well-planned community.

6.—Another characteristic of the broad nature of town planning is illustrated in its economic, legal, and administrative aspects. Town planning cannot proceed a step without counting the cost. The final conundrum is paying the bills. This often is the check to town planning advance; and yet nothing is more certain than the fact that town planning pays both in direct and indirect ways. Certain features are indispensable for a community, such, for example, as suitable main thoroughfares, land for parks and other public uses, proper provision for railroads, including location of rights of way, passenger and freight stations and railroad yards; also, public building sites, especially well-located land for schools, and areas for housing with adequate and pleasant home grounds and gardens. All these features a town must have, if it is to endure and prosper. The only choice is whether they are to be obtained early with the initial planning when land is cheap and conditions controllable, or later when land is expensive and conditions are fixed and more or less unchangeable. Moreover, a town has only a choice usually of the form of its expenditures. If it does not provide these essential features for town life, the people must pay an equal or greater sum in other and less satisfactory ways. An examination of such subjects, as traffic regulation, recreation, education, housing, showing comparative conditions and comparative costs in various towns would confirm this statement.

7.—Town planning has its legal side and administrative machinery. It involves an understanding of the rights of property compared with the rights of persons; the rights of single individuals compared with the rights of the group. Furthermore, there is the administrative machinery for carrying out the town plan. How to get the thing done is the vital issue. The town plan must be put in action. It must work. Indispensable as is the paper stage, it is the first stage only. The question must be asked, by what means can the plan and its program be carried out?

Broad requirements such as those cited as indispensable for town planning call not only for the usual professional skill, but also for imagination, for the

approach that is characteristic of the artist, for one of creative mind, for the point of view that shows power to grasp, to simplify, to express social ideals in terms of a better ordered, less wasteful, more satisfying, and more beautiful town. Mr. Raymond Unwin, the distinguished English town planner, states this particularly well in an address,* which might have the title "The Artist and the Practical Man", in which he says:

"Hitherto the work of town planning has suffered for want of clearer understanding, even on the part of those well versed in the subject, of the difference of faculties and methods needed for success. If the practical man has sometimes thought that complete mastery of the science of the subject would suffice to enable him to practise that which is as much an art as a science, it must be admitted that the artist has at times also imagined that his training and his art have forthwith qualified him to become a planner of towns, forgetting that this particular art is based on an extensive science, which must be at least understood. * * * What the artist specially needs is a sympathetic insight into all the relationships of city life, a realization of the reactions which take place between the city environment and the human society which it clothes and expresses. He needs, in fact, that particular range of knowledge which will enable his imagination to picture the city as it might be, to see the life of the people going forward in it, to see all the different parts and functions in their true relation. He needs this that he may be able to study his vision effectively and mould it to meet the realized conditions, or modify it to avoid the apprehended difficulties. The kind of knowledge needed is extensive rather than intensive; for there must be maintained a degree of detachment from the details of the problem if the city and the life of the city are to be seen fairly and seen whole. The town designer must prepare his imagination for this work by watching and thinking over the phases of city life; meditating on their comparative manifestations in many towns; entering sympathetically into the needs and limitations, musing all the time on visions of how work might be made more efficient and town life more pleasant."

With this statement of the nature of town planning, the part played by the various professions usually employed in carrying out the development work of a community may be considered, as well as the requirements placed on the engineer, the architect, and the landscape architect, and their relation to the town planning authority.

First of all is the engineer. He naturally is the man to come on the ground at the very beginning. The basic feature of his work is the topographical survey. Although it has often been thought so, the topographical survey is by no means the whole of the survey of local conditions. It is, however, the most indispensable element. A recent writer has paid tribute to the part played by the surveyor in the development of the Nation, pointing out that the list of surveyors includes such widely different and gifted men as Washington, Daniel Boone, Andrew Jackson, Thoreau, and Lincoln. A surveyor blazes a trail, he defines boundaries, he must be undismayed by obstacles and barriers. Partly because of the fact that the engineer in the form of the surveyor is first on the ground, much town planning has heretofore been done by the engineer. Usually, however, until recently the engineer has brought to town planning only engineering knowledge and not town

* "The Architect and His City", printed by permission of the Royal Inst. of British Architects, in *Journal* of The Town Planning Inst., December, 1925, p. 36.

planning knowledge, and it has frequently been specialized engineering at that, rather than fundamental engineering. For with the great development that has come in engineering science, the field of engineering has been divided and subdivided and specialized, so that to-day the type of engineer that formerly was so all-embracing, so well known, and still so well respected, termed the Civil Engineer, has become in practice largely a thing of the past.

In a valuable address on "The Engineer and the Town Plan", by the late James Ewing, delivered before the City Planning Division of the Society at Montreal, Que., Canada, on October 15, 1925,* Mr. Ewing showed how mere engineering, even when most skillful, often fails in town development unless somehow it can be guided and controlled by town planning knowledge. He illustrated his statements by an analysis of what too frequently has been the work of engineers in connection with city streets, bridges, railroads, etc., and pointed out that these public works are too often monuments of engineering enterprise and skill, and at the same time lasting memorials of wrong judgment and radically bad planning from the standpoint of a community or of National development. He went further—he showed convincingly how the natural resources of Canada have been wasted and the Nation burdened with debt because the engineering was so often done from a narrow engineering viewpoint, in place of a broad planning point of view.

As the trained and experienced engineer should, of course, be entrusted with the works of engineering in building a town, so should the trained and experienced architect be entrusted with works of architecture. Community development, however, involves something more than the construction or mere assembling of engineering, architectural, and landscape architectural works, meritorious as these may be. It involves the regulation and control of all these constructions in the interest of the whole, for only in this way can the maximum of convenience, good construction, and right appearance be secured.

The architect's work as architect, requires provision for stability, convenience, and beauty, also a nice regard for cost; but a building, be it large or small, might meet all these provisions and yet fail when tested as to its place in community development, because of a lack of proper relation to other architectural works, to engineering, or to landscape architecture, or to some element or feature of the town plan itself. Therefore, the architectural works of a town should be done, if possible, by an architect who himself understands town planning, or else his work should be directed and approved from the town planning point of view, represented by a qualified individual or group of individuals possessed of a variety of town planning knowledge and experience. For a happy fusing of all the complex works of the various professions is the desired goal. Co-ordination is essential.

There is little to be gained by attempting to determine whether the engineer, the architect, or the landscape architect is better fitted for the work of town planning. Each has his own peculiar advantages and limitations. In the United States, the landscape architect has taken a prominent part in

* See p. 303; also *Journal*, Town Planning Inst. of Canada, October, 1925, p. 8.

town planning for two reasons: First, for several generations the profession has had the good fortune of having a leading firm of gifted men, with high ideals of professional work and public service and a developed social sense, who, in turn, have trained many younger men. This firm has put into landscape design more of the spirit and requirements of town planning than have yet manifested themselves in any other direction. It is true the landscape architect has a natural call toward town planning. He thinks always of buildings in relation to site and topography. He is concerned with their approaches, their background, and their environment. He has regard for natural features of beauty, and their value for recreation or for other important town uses. He is trained in broad design and composition, and the arrangement in convenient, orderly, and agreeable fashion of structures often primarily the work of engineers or architects. The landscape architect thus turns naturally from gardens to parks, from parks to civic centers and streets, from streets to suburban development and land subdivision, and from land subdivision to town planning. There is an almost inevitable evolution, step by step, not only in the professional technique of planning, but also in weighing the claims of all the various elements of convenient arrangement, requirements of construction, cost, and appearance, so that the result may be as nearly right as it can be, from many points of view.

A second reason for the high place occupied by the landscape architect in town planning, a reason that is at once both an effect and a cause, is that the most complete course of specialized training in town planning offered by an American university is most closely related to the advanced technical courses and research work in landscape architecture. These courses of instruction, however, are equally open to engineers and architects, and have been taken by many men already well trained in these professions. One of the most valuable discussions of this phase of the subject is contained in the series of articles by Mr. Thomas Adams.* The gist of the matter is well expressed by Mr. Adams in these words:

"The appointment of a special visiting committee of the School of Landscape Architecture at Harvard may lead to interesting developments in connection with the teaching of town planning, which in Massachusetts includes city planning. The appointment of the committee should promote further co-operation between the schools of architecture and landscape architecture rather than tend to create any line of cleavage between them. That is agreed to be highly desirable. Both the architect and the landscape architect who intend to specialize in town planning need a higher degree of training in the principles and methods of civic design and more understanding of the reciprocal relations of all the factors in city and town development. The landscape architect *per se* is no more a town planner than the architect, or the architect than the engineer—and no member of any of the three professions has a claim to be a town planner in a more comprehensive sense than the other.

"A member of each profession has the knowledge and qualifications needed as a foundation for making the town planner, but specialized training is needed to be superimposed upon that foundation to make either the architect town planner, the landscape architect town planner, or the engineer town planner. The relative degree of importance of either in town planning will

* *Journal*, Am. Inst. of Architects, Vol. 10, 1922, p. 101.

be a matter of personality. Until recently the field of modern town planning in America has appeared to be the specialty of the landscape architect, and no one will question the fact that some distinguished landscape men have given reality to any special claims their group may have appeared to have for the cultivation of that field. Moreover, in Harvard and elsewhere, special teaching in phases of town planning to landscape students has given further stability to these claims. Similar teaching has not been given, or at any rate, to anything like the same extent, to architects and engineers."

Town planning, as far as the special subject of this paper is concerned, finally simmers down very largely to the relation between design and construction, and the methods by which design can be translated successfully into construction, keeping always in view the element of cost and the legal authority with which to proceed. There must be wider training in design of those who have to do primarily with construction, and a larger experience in construction of those who have to do primarily with design. The town plan itself must show an orderly distribution of all its parts, with a nice regard for the practical requirements of each part, and its esthetic potentialities; and there must be foresight and vision as to what is involved in the social and civic life of the community in the execution of the plan. Detail planning must follow general planning, and again the town planning point of view must prevail. Finally, actual construction takes its place upon the ground in the public utilities, or raises its head above the ground in architectural or engineering structures. Here, once more, the local purposes must be served, but with these local purposes, if there is knowledge, skill, good sense, and co-operation, the spirit of town planning enterprises may be successfully invoked to give, under all the circumstances, the best possible results. In human affairs which have the variety and complexity of town planning, perfection is not possible, but a much higher degree of public convenience and private enjoyment may be secured with a considerable saving of costs, and a distinctly better quality of urban beauty obtained, if a sound method of organization is used in harmonizing the work of the various professions with town planning principles and ideals.

DISCUSSION

JOHN W. ALVORD,* M. AM. SOC. C. E. (by letter).—One cannot read Mr. Nolen's inspiring paper without broadening one's conception of professional service to the public, and receiving, as well, a stimulating vision of the fuller possibilities of community life.

Whatever prevents men from unnecessarily crowding, interfering, and distressing one another and frees them from hindrances to the better expression of their varied and constructive activities, and, finally, inspires them with ideals of individual and community welfare, is, indeed, a great forward movement.

Thus, the writer would define town and city planning as those foresights in physical adjustment necessary for the fullest expression of the individual in his community life.

The Engineering, Architectural, and Landscape Engineering Professions, as well as other kindred fields of "practical artistry" and specialized service, have a common aim and purpose in uplifting service to the public; and it is singular that engineers, the professional descendants of Leonardo da Vinci, Michel Angelo, and many others, who were at once both painters, sculptors, architects, and engineers, should have grown to think of these fields as separated callings, to their distinct disadvantage, when as a matter of fact they are only different and overlapping aspects of the one broad field of creative design.

This was made amusingly evident some years ago when the architects and engineers attempted to define their several callings for the purpose of legal regulation. It simply could not be done.

The work of successful city planning, as Mr. Nolen has shown, requires a broad insight into every class of creative design, and foresight from the intensely practical up to the artistic and spiritual. It calls for broad-minded men, not only with especial kinds of training, but having above all sympathetic insight into all other fields of human endeavor, besides the one in which they have been especially trained. Specialization tends to obscure these broader requirements, and it goes without saying that to overcome its narrowing concentration is a cultural success that is attained by but few men who, with great diligence, make the attempt to meet these requirements.

It was the writer's good fortune, during the World War, to spend a year with the United States Housing Corporation at Washington, D. C., where town planners, architects, and engineers were closely co-ordinated in nearly one hundred housing developments. Early in that experience the impression on the writer was, not the separateness of these three callings, but, at the bottom, the remarkable similarity. One could hardly call them partly overlapping, in any broad review at least. They were, perhaps, best described as uni-central in purpose, with varying aspects along their outer boundaries, or in matters of detail function.

Unity of general purpose and fitness of each co-ordinating function, with variety and harmony in detailed method, as practical opportunity is afforded,

* Cons. Engr. (Alvord, Burdick & Howson), Chicago, Ill.

is the keynote of good town planning. Mr. Nolen has admirably pointed out some of the necessary details for the application of these principles.

William T. Lyle,* M. Am. Soc. C. E. (by letter).—"Town Planning and Its Relation to the Professions Involved" is undergoing a great change, owing to the changing nature of towns themselves. This development in its regional character requires the solution of new problems, and emphasizes the desirability of uniting the professions in developing a city planner.

Were the problem one of *de novo* design as in the case of Washington, D. C., the city planner with training as a landscape architect would be far better qualified, other things being equal, than the planner from the ranks of architects or engineers, even if he should find it necessary to consult with the others.

There was a time, not many years ago, when city planning was concerned with the laying out of the "city beautiful"—when the design and construction of an elaborate park system was supposed to be the chief element in municipal improvement. There was also a time, even more recently, when city planning consisted principally in the building of a showy civic center.

Those days have passed. People now realize the magnitude and comprehensiveness of a task which involves all the physical elements of the city's conformation, not only of the city proper, but also of the region about it from which it draws for its needs and to which it ministers as a center of supply and distribution.

The shifting emphasis has been brought about by the change in the towns and cities themselves. Towns laid out for the conditions of Colonial times, with streets of more than ample width for the requirements of the Eighteenth and even the Nineteenth Century, have grown to so great a degree that those same streets are inadequate to the demands of even a single office building. Railroads once properly located on the outskirts of small towns now find themselves in the very heart of large cities with grade crossings seriously menacing public safety, and seriously hampered in operation by reduced speed of trains and increased expense in the protection of crossings. Shipping facilities once equal to every demand made on them now find themselves hampered by restricted pierhead and bulkhead lines and by inadequate railroad terminal co-operation. In many cities the housing problem is acute; in many, also, the problem of local passenger transportation remains unsolved; and in all the problem of street traffic is one of the greatest difficulty.

Owing to the growth and development of towns and cities the application of the engineer's art in the making of the city plan is becoming vastly more important. It is no longer sufficient for the city planner to confine his attention to the laying out of streets, parks, and civic centers. He must also consider problems of traffic control which have baffled solution; problems of how and to what extent streets can be widened; problems of river and harbor design and improvement; problems of terminals and bridges, together with their approaches; problems of track elevation and depression, trackage elimination, union stations, loop lines, and classification yards.

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

City planning is intensely urgent and practical, and the author has pointed out the demand for talents of great versatility. All these talents have not been found in one profession—not in the civil engineer, certainly, the architect, or the landscape architect. With the shifting of emphasis, from the esthetic to the architectural, and lastly to the utilitarian, such talents as are found in the engineer are coming more and more into demand.

The time has arrived for the development of the city planning engineer, a man in whom the work of city planning can "head up" and under whose regulation and co-ordinating guidance the great and diversified task can proceed.

He must be primarily a man of the engineering type, capable of solving those pressing modern problems which are baffling solution. He must be a man trained in co-ordinative architecture as far as it pertains to the grouping and arrangement of buildings, in street design, and in the general principles of park design; but above all in those economic and constructional problems due to restriction and congestion in traffic and transportation. Such planners have not yet been developed, except possibly a few in the school of practical experience.

In view of the great demand for broad-gauge men for the solution of existing difficulties in congested cities, it is time for a few of the larger universities to turn their attention to the development of engineers of this type, men not only versed in the general principles of city planning architecture, but qualified to plan for the relief of modern congestion on the firm practical basis of economic municipal wisdom.

Thomas Adams,* Esq.—It would have been interesting to have heard more of Mr. Nolen's reasons for difficulties in regard to density and garages. It is the discussion of the difficulties found in special cases of this kind that gives a conception of the methods for treating these problems under ordinary conditions. Similarly, useful would have been some indication of rents or costs in relation to turnover, because the value of Mariemont, Ohio, as an object lesson of development, is in proportion as it demonstrates how it can be carried out elsewhere.

The city planner and the engineer have an obligation to society to show that whereas what they carry out is more beautiful and more idealistic, it is also, in its application elsewhere, more profitable from a financial point of view.

There is a query on one point mentioned by Mr. Nolen. What is wrong with the modern city? It is not that it is too big but that it is wrongly planned and wrongly developed. It is no worse to live in the crowded districts of Manhattan or of London with their congested streets than in the crowded tenements in a small city or village. What is wrong is not that cities are too big but that there is an improper balance in their growth. If the garden city or Mariemont could be repeated a thousand times where there

* Gen. Director of Plans and Surveys, Regional Plan of New York and Its Environs, New York, N. Y.

was ample space and ample opportunity for light and air, there would be no need to worry about the size of the area that would be covered by a group of such communities.

The speaker envies Mr. Nolen and his associates in the opportunity presented by this plan; the more so because one or two of his failures have been exactly in cases of this kind. No effort was spared to make a plan that seemed satisfactory and that would be the occasion of considerable satisfaction; and yet, owing to the wrong architect being employed or to some form of bad management, it was anything but a success. The necessity for collaboration between the landscape architect, the architect, and the engineer is demonstrated in such instances.

Sometimes the controlling management, in the interests of false economy, has destroyed the opportunity to make a success. For instance, the business administration should not be under the city planner, but neither should the city planner be under the business administration. They should be two separate organizations, dovetailing together, working together, with possibly a final arbitrator in case of differences; but there should be no subordination of the technical expert in matters of design.

As to the plan—no one is in a position to make intelligent criticisms of a plan unless he has gone through the same experience and obtained the same knowledge of the site as the man who has made the plan. There can be only congratulations for Mr. Nolen, not only on the opportunity he has had at Mariemont, but on the apparent technical perfection of his work.

All agree as to the desirability for more collaboration between the landscape architect, the engineer, and the architect. How can this be brought about? The meeting at which this paper was presented, was held under the auspices of the American City Planning Institute in collaboration with the City Planning Division of the Society. There also exists a group of the American Institute of Architects who are interested in something that is different in name but really is the same thing—"community planning." Another effort should be made to bring about the needed collaboration. That may be accomplished somewhat in the way it has been done in England in framing the Constitution of the British Town Planning Institute. The basis of that Constitution is this: A person may be a full member of the British Town Planning Institute if he is a full member of the Institution of Architects, the Institution of Civil Engineers, or the Surveyor's Institution; or he may be a legal member if he is a qualified lawyer. However, he must have a degree in one of these four standard professions before he can be a member of the City Planning Institute. This is the general rule—but provision is made for election of distinguished men who have special qualifications, other than membership of the four basic Institutes or Societies.

The American City Planning Institute should be re-organized and developed on a strictly professional basis, with due regard to bringing in the architect, the engineer, the landscape architect, the lawyer, and, on a few exceptional occasions, the sociologist and others who are especially interested and who have already gained a certain standing. There are greater

tasks awaiting the city planner in the future than in the past, but in order to deal with them he must be better equipped with technical knowledge and better organized.

EDWARD M. BASSETT,* ESQ.—Mr. Adams has referred to plans that he had helped to make and that he had seen others help to make which, after a number of years, were not carried out. One reason for this is because the elements that were carried out were not town planning; they were elements that had no security, and a change of management obliterated them, or the exploitation by private individuals, not regulated by law, allowed them to disappear.

A demonstration has been given of planning of the community and for the community, but there has been no demonstration of planning by the community. In the long run, communities are going to be the result of planning by the community. At present, such planning is bad and incomplete. The powers of the community are not sufficiently cast into laws and made available so that planning by the community is effective. That gives all the greater place to those who will bring about, as many have helped to bring about, these demonstrations of private planning.

Some one will say, "Mr. Bassett is injecting his usual lawyer views into this subject", but this is not especially the position of the lawyer. Lawyers have no more to do with statutes than the architect, the engineer, the landscape man, or the ordinary citizen interested in good town planning; yet perhaps lawyers are a little more apt to come into the exact definition of these things, or make an effort to. The lawyer has no great place in town planning—he is the least of those who can really contribute; but possibly sometimes the lawyer can help to define, and if the definition is at all sound those who do the real work of town planning—the engineers, the architects, the landscape men, the social philosophers—will be better able to collaborate.

In nearly all discussions of town planning, various qualities or exhibitions of such planning are presented. What is town planning? By this is not meant the objects of town planning or the results of town planning. Town planning is the determination of the legal quality of land areas for public purposes. That is a simple and short statement that does not include the objects, or results, or the elements that constitute the subject, or describe what the different ones who do the town planning should contribute. To "determine the legal qualities of land areas" requires the engineer, the architect, the landscape man, the social expert, and the housing expert, and the more they know about it the better they can do the town planning.

After twenty or thirty years, when the Town of Mariemont, Ohio, is turned over to the public authorities as a community to be run by the community—five or ten years after that date—what will be left of the town planning? It will be the legal quality of the land areas, namely, the streets, the parks, the sewer lines, the gas lines, the transit lines, the sites for public buildings—it will be those things that relate to the legal quality of the land. Nothing will be left regarding the type of house that has to be built when one burns down, except the zoning that has become fixed on the land, and that zoning is the

* Director, Legal Div., Regional Plan of New York and Its Environs, New York, N. Y.

impressing of a quality by law on the land. The color of the houses, the design of the houses, will all have disappeared, because it cannot be expressed by valid law.

The laying out of the golf course in a park is landscape work, but it is not town planning. The fixing of the value of houses that must be constructed by private restrictions is very valuable in organization, but it is not town planning, and when the Town of Mariemont is turned over that will disappear. It may be that there are private restrictions that will continue in projects such as Palos Verdes and Mariemont. If those private restrictions are kept up, or the rule under private restrictions of a corporation that will dominate the community, then there is something that will, for a time, perpetuate itself, but it will not perpetuate itself forever—only for 15 to 30 years. In the long run communities are going to disregard those private restrictions, and that private control, unless all the land is owned by one corporation, will evaporate. In fifty years from now, the plan of Mariemont is going to be the quality that has been impressed on streets and parks, on waterways and water edges, on transit lines, and the zoning of private land; for proper town planning relates to private property as much as to public property because a legal quality can be impressed on private land.

If town planning is impressing the legal quality on land areas for public purposes, after that is done there is the town plan; the architect has his later work to do; the engineer has his later work to do in the street surfaces, the making of water-works and the gas and sewerage systems; the landscape architect has his work to do in laying out the park surfaces and the location of civic centers. The man who combines all those qualities is the ideal city planner, but they all co-operate in carrying out the intention of the plan—and the plan is the impression of the legal quality of land areas for public purposes.

J. E. Willoughby,* M. Am. Soc. C. E. (by letter).—Much of the town planning for American communities will be in the nature of a re-adjustment of existing features to conform to present and prospective conveniences and needs of the people and to provide for the co-ordination of those re-adjusted features to growth outward from the exterior boundaries of the city. Existing property rights will be affected. The individual loss sustained by the property owners as a result of the adjustment must be resolved into terms of money and paid for by the community. The failure of many experts engaged in town planning to recognize that obligation defeats the actual accomplishment of many worthy projects.

One of the more common lapses in this connection is the assumption that the steam and electric railways can be moved at the caprice of the planner and that the property owned and the rights enjoyed by those railways can be converted to public use without just compensation. The prosperity of all American commercial communities is dependent on rail transportation. Any plan that impairs the ability of the railways to serve best the needs, present and future, of the people is to be condemned as being contrary to the prin-

* Chf. Engr., A. C. L. R. R., Wilmington, N. C.

ciples and ideals of town planning. The views of those engineers who are responsible for rail transport, as to what facilities and non-interferences by other forms of transport will be needed for the rail service, are worthy of consideration as a major requisite.

NOULAN CAUCHON,* ESQ.—The questions raised in this excellent paper are of much interest to engineers engaged in city planning. The experiences of the Town Planning Institute of Canada in general substantiate the author's conclusions. The technical qualifications required for its members are similar to those of the Town Planning Institute of Great Britain, the object being to focus technical knowledge on the question of public welfare. Thus, the engineer, surveyor, landscape architect, or lawyer who applies for membership is supposed to be qualified in or recognized by his own profession.

In some quarters there is a feeling that the architect will supersede the engineer, or *vice versa,* but the experience of the Town Planning Institute of Canada does not support this. Any city planner, no matter what his primary profession, will recognize the great advantage of collaboration with specialists in other fields. Even if the public does not recognize the need of special training, by all means the specialist should recognize his brother. For example, no one should know better than the engineer that he is not an architect, or *vice versa.*

The City of Ottawa has a Town Planning Commission of which the speaker is Chairman and Technical Adviser. Although it is an official organization, under the laws of the Province of Ontario, and has city officials as members, its powers are purely advisory and its recommendations are subject even to disapproval by the City Council. As a result, sometimes regulations and permits are made effective by Council actions, which are clearly contrary to ideal city planning. This illustrates the necessity of giving town planning the authority of law, so that it will become the will of the community and not merely the advice of an expert.

The objects of city planning, particularly of zoning, are not to crystallize, but to stabilize. The successful plan must be made elastic. Safeguards must be taken, so that changes, especially extensive changes, cannot be made overnight. In Canada, and it is hoped, in the United States, there is good reason to think that the future holds favorable promise for the effective work of the town planning engineer.

JOHN NOLEN,† M. AM. SOC. C. E. (by letter).—Each of the discussors has made comment of value, which becomes evident to any one who will follow the discussion through. In this final summing up, the main points will be mentioned.

The subject "Town Planning and Its Relation to the Professions Involved" unquestionably calls for the training, technical skill, and experience usually spoken of as engineering, architecture, and landscape architecture. In all important work these services should be rendered, if possible, by a group of

* Cons. Engr., Ottawa, Ont., Canada.

† City Planner, Cambridge, Mass.

individuals. In some cases, however, it is not practical, because of cost or other reasons, to secure a complete group, and, in other instances, such a group may not be essential.

The employment of a single office, primarily that of an engineer, an architect, or a landscape architect, does not mean that only the training, skill or experience of one profession is employed, because an office properly equipped for city planning as a regular part of its work almost invariably has on its staff men of various training and experience—engineers, architects, and landscape architects.

Several definitions of city planning have been given, which are helpful as revealing various and somewhat contrasting points of view. For example, Mr. Alvord defines town and city planning as "those foresights in physical adjustment necessary for the fullest expression of the individual in his community life"; again, he states:

"Unity of general purpose and fitness of each co-ordinating function, with variety and harmony in detailed method, as practical opportunity is afforded, is the keynote of good town planning."

In contrast with this is the statement of Mr. Bassett. He asks,

"What is town planning? By this is not meant the objects of town planning or the results of town planning. Town planning is the determination of the legal quality of land areas for public purposes."

This statement may be true enough if the limitations which Mr. Bassett has himself placed upon it are kept in mind. It is apt, however, to be confusing, and does not appear to be a correct definition of "town planning". It is of the utmost importance that the planning itself should be properly done by qualified individuals before this legal quality is fixed in the land. The supreme element in city planning is that of design. The trouble to-day in existing cities is largely because the legal quality that exists in the land is not the right legal quality in the right land. Streets, for example, are wrongly located, and not of proper width. Railroads are improperly placed. There is a great lack of the necessary open spaces. Foresight has not been exercised in selecting sites for civic buildings, public schools, etc., and the wrong use or over-use of private land has been established. Mr. Bassett speaks also of the change in the use of land. This may be very desirable, for the change may represent changing requirements.

Professor Lyle refers to what might be called the evolution of modern city planning. He mentions the "city beautiful" idea, and the emphasis on civic centers; also the co-ordinate problems of traffic control; street widening; river and harbor improvement; and railroad terminal and track changes. All this illustrates the comprehensive character of modern city planning and the necessity for a broad point of view, not only of the city itself, but of a region.

Regarding the important question of the organization of city planning, and the inter-relation of the various planning professions involved, one of the best statements was from Mr. Alvord, in connection with "the United States Housing Corporation at Washington, D. C., where town planners, architects, and

engineers were closely co-ordinated in nearly one hundred housing developments." The impression made on him was, "not the separateness of these three callings, but, at the bottom, the remarkable similarity."

Mr. Adams also drew from his wide knowledge and experience in this field when he pointed out that effort should be made to bring about the needed collaboration, mentioning the British Town Planning Institute, to become a member of which a person must be a full member of the Institution of Architects, of the Institution of Civil Engineers, of the Surveyor's Institution, or a qualified lawyer.

The same idea is well brought out in the discussion of Mr. Cauchon, who refers directly to the practice of the Town Planning Institute of Canada.

In conclusion, perhaps the most constructive suggestion to come out of this discussion is the final word from Mr. Adams', in which he states:

"The American City Planning Institute should be re-organized and developed on a strictly professional basis, with due regard to bringing in the architect, the engineer, the landscape architect, the lawyer, and, on a few exceptional occasions, the sociologist and others who are especially interested and who have already gained a certain standing. There are greater tasks awaiting the city planner in the future than in the past, but in order to deal with them he must be better equipped with technical knowledge and better organized."

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1644

THE DEVELOPMENT OF MARIEMONT, OHIO*

By Frederic H. Fay,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles Wellford Leavitt and Frederic H. Fay.

Mariemont, Ohio, a town in the making, situated just outside the city limits of Cincinnati, is a practical example of the application of the principles of community development laid down by Mr. John Nolen,‡ M. Am. Soc. C. E., and is the result of the joint effort of men representing various professions. At Mariemont, which is patterned after the English garden cities, American methods, combined with modern city planning ideas, good architecture, and sound engineering practices, are creating a community which is unique in the history of town planning in the United States.

Most municipalities are founded by men; Mariemont's founder is a woman, Mrs. Mary M. Emery, one of Cincinnati's best loved citizens. It is the result of many years of planning by herself and her associates, and is only one of the numerous ways in which she is making her wealth of real and lasting service. The new town of Mariemont, however, is not a philanthropy nor in any way paternalistic; it is not merely an undertaking to build more houses; it is a real estate development on normal American lines, except that profits are restricted to a low figure.

Purpose

Mariemont is intended primarily as a place of residence for families of widely different economic standing, and especially for wage earners. It does not provide homes for the very poor; experience thus far has shown the prac-

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

† Cons. Engr. (Fay, Spofford, and Thorndike), Boston, Mass.

‡ "Town Planning and Its Relations to the Professions Involved," p. 721.

tical impossibility of providing new homes for people of the lowest economic scale. Mariemont is an outstanding example of the fact that, with careful planning, and with the savings in cost resulting from large-scale construction, it is possible to provide practical, convenient, and well-made homes, amid ideal surroundings for people of moderate means. It is not a laboratory for sociological experiments in the problem of housing. It does not follow the English plan of co-partnership building and ownership. People in America are still too individualistic in their attitude and action to take readily to co-operative housing schemes.

A National Exemplar

At Mariemont, individual home builders, and projectors of towns and subdivisions near great cities, will find a practical demonstration, not only of careful planning on what are believed to be correct city planning principles, but also of the advantages of modern methods in town building and of the value of beauty both in placing and designing a home. In these respects Mariemont may be called "a National exemplar".

Town Site

The site of Mariemont, about nine miles east of the business center of Cincinnati, is a slightly undulating plain, formerly rich farms and woodland, extending from the base of the steep slopes of Indian Hill, which protects it from the sweep of north winds, to the edge of a wooded bluff overlooking the Little Miami River and its broad level valley 100 ft. below. Across the valley may be seen other Ohio hills and in the far distance the rugged Kentucky hills on the southerly bank of the Ohio River. As the prevailing winds in this locality are from the southwest, there are always, in summer, cooling breezes up the Little Miami Valley over the Mariemont plain. In its landscape setting and other natural features the site is most attractive as a place of residence.

General Description

The general plan of development of Mariemont, covering a tract of about 420 acres, provides for a complete town to include homes for 6 000 to 7 000 people (Fig. 1). Future development of land bordering this tract will probably result in Mariemont becoming ultimately the nucleus of a community of more than double this population. In planning the town, therefore, provision has necessarily been made for meeting the needs of the larger community.

The town is essentially residential. For this reason there are to be no industries directly within the village. Two industrial sections with rail and highway connections are provided, however, one at the westerly end of the property and the other along the banks of the river below the bluff. These are situated so that, although they lie within easy distance of the residential part of the town, there are natural barriers which make them wholly separate and distinct.

Fig. 1.—Layout of Central Portion of Mariemont, Ohio.

A complete town means that all public and business buildings, all parks, playgrounds, and recreational features, and all utilities necessary and desirable for a fully developed municipality are being provided at the outset, together with the building of residences. When construction operations now under way are completed, the town will be fully equipped, everything in it being new and up to date. In its development, however, care has been taken to preserve trees and other natural objects of beauty so that when finished the town will have the appearance and possess the advantages of a long settled community.

The design of Mariemont provides for a town center with a village green, similar to those found in many English country towns. In and around the town center are grouped the principal buildings, the town hall, library, the community building, post office, hotel, bank, and theatre, together with stores and a public market, all so situated as to be within easy walking distance of the residential parts of the town.

The street plan consists of major highways radiating from the town center and secondary residential streets, with service lanes or alleys at the rear of lots. One major thoroughfare from Cincinnati to the east passes through the town center. Along the edge of the bluff, overlooking the river, is a boulevard extending the entire length of the property, a distance of 1½ miles. This boulevard, which will eventually be linked with Cincinnati's Boulevard System, serves as a by-pass route for through traffic around the town center. In all, there are five main thoroughfares through the village to avoid congestion and facilitate convenient circulation.

A feature of the development is the open-air concourse immediately south of the town center with which it is connected by a main avenue. Situated on the edge of the bluff, the concourse preserves as a public privilege the beautiful view of the Little Miami Valley and the distant hills. It provides also an ideal place for public gatherings, being large enough to accommodate half the entire population of the town.

Community Buildings.—The community church (Fig. 2), the first structure to be built, is a memorial to the early settlers. It is constructed of native stone in the Norman style and is a copy of Stokes-Poges, in England, the church in which William Penn and his ancestors worshipped and in the yard of which Gray wrote his famous "Elegy". The huge hand-hewn oak beams forming its rafters were taken from an old mill which had stood in the vicinity for more than a century. The roof is to be of stone brought from England. In its setting among old and rugged trees, and adjoining a small century-old graveyard, this non-sectarian community church possesses a charm and an appearance of age which are noteworthy.

The first of the several projected school buildings, a grade school of eight rooms and a kindergarten (Fig. 3), is already in use. It is a brick structure designed and equipped as an example of best modern practice in school building.

On the slope of Indian Hill overlooking the town Mrs. Emery is providing, as a memorial to her late husband, a hospital adjoining which are a con-

valescent home and work shops and a demonstration farm; these buildings are grouped in a section of the town known as Resthaven.

Recreation Facilities.—The recreation facilities include playgrounds, a stadium, athletic field, and tennis courts, together with parks of different types and scenic reservations for public use on the bluff above the river. Of the park areas which comprise a total of more than 50 acres, Dogwood Park deserves particular mention. It is in a deep ravine through which flows a small stream where a lagoon is to be constructed. The park site will be preserved in all its natural, rugged beauty.

Housing.—Mariemont is chiefly of interest as a housing development. To meet the needs of those who can afford only low rentals, there are many small flats or apartments, some even of a single room, and also a number of "group-houses", that is, houses attractively grouped together in a row, as shown in Fig. 4. For residents of more ample means, duplex houses and single houses are being built, as shown in Figs. 5 and 6.

Every residence, from the smallest to the most pretentious, has been skillfully planned and well constructed of durable materials, mostly brick and stone. There are no dark rooms in Mariemont. Every home is provided with running water, bath, and other sanitary facilities, electric light, telephone, and natural gas for cooking. The larger residences, as well as all public buildings, stores, and apartments in and around the town center, are to be heated from a central steam-heating station. Especial care has been taken that no slum district shall ever develop in Mariemont, no matter how thickly populated it becomes. The ample yards in the rear of all habitations, the building setbacks, and the many parks, public squares, and allotment gardens, guarantee perpetual pure air and sunshine. The lots are large and the space between buildings is fixed. Except for a certain few buildings, principally stores and apartments around the town center, no building more than two stories in height is to be erected in Mariemont.

Architectural Work

The architectural planning of Mariemont has been apportioned among twenty-six architects of high standing in Cincinnati, Boston, Mass., New York, N. Y., and Philadelphia, Pa. The result is a pleasing variety and treatment of architectural design, both of residential structures and of community buildings. There is complete absence of the monotony which frequently characterizes large-scale community development. The architectural work has been co-ordinated so as to obtain attractive results in accordance with the carefully thought out plan of the projectors and of their expert advisers. Mariemont will always be an interesting museum of American architecture, for probably nowhere else in the United States is there a comparable display of the best work of contemporary architects covering such a wide range of building types.

The visitor to Mariemont is impressed with the work of the architect and of the landscape architect. Less noticeable perhaps to the casual observer, but of no less fundamental importance, are the results accomplished by the town planner and the engineer.

FIG. 2.—THE COMMUNITY CHURCH, MARIEMONT, OHIO.

FIG. 3.—THE GRADE SCHOOL, DALE PARK, MARIEMONT, OHIO.

FIG. 4.—TYPICAL GROUP HOUSES, MARIEMONT, OHIO.

FIG. 5.—TYPE OF DUPLEX HOUSES, MARIEMONT, OHIO.

FIG. 6.—SINGLE HOUSES OF SIX AND SEVEN ROOMS, MARIEMONT, OHIO

FIG. 7.—BOILER-HOUSE FOR CENTRAL STATION HEATING SYSTEM, MARIEMONT, OHIO.

Town Planning

As an example of town planning Mariemont may well be called "a National exemplar" and the broad developments which are now taking definite shape are the result of long, careful, and able town planning. The worth of town planning can best be appreciated by detailed study of the layout both on plan and in the field. Success has met the attempt to conserve the natural beauty of the site and to provide a community which shall be not only attractive but also of maximum usefulness to its citizens.

Engineering

Of particular interest to, and within the province of, the engineer are the utilities of the town, most of which are not seen and may not be appreciated by the visitor. The building of an entire new town on practically unoccupied territory provided an exceptional opportunity to plan a comprehensive scheme for the underground utilities and to carry out their construction in advance of other constructions, including street surfacing.

Utilities

Included in the engineering work are the design and supervision of construction of the municipal underground utilities, the water supply, sewerage, drainage, and heating systems; supervision of the installation of the underground facilities of public service corporations, including gas, telephone, and electrical distribution systems; and the design and supervision of street pavements.

Water Supply.—The water supply is obtained through two entirely independent supply mains connecting with the high-pressure water system of the City of Cincinnati. The use of duplicate mains insures a continuous supply of water. Near the junction of each supply main with the Mariemont distribution system, a detector meter is installed to measure the volume of water supplied. The distribution system for the town is built as a "gridiron" with practically no "dead ends", thus assuring a dependable and uninterrupted supply of water both for fire-protection purposes and for individual use. The water mains are laid beneath the sidewalks on one side of the street only, and are of heavy cast-iron pipe. Hydrants are provided at intervals of 300 to 350 ft. This hydrant spacing makes the use of pumping engines unnecessary, even in a serious fire, owing to the high water pressure of 150 lb. per sq. in. available at the hydrants. Individual water services of brass pipe have been constructed from water mains to all lot lines so that it is unnecessary to tear up streets whenever houses are built.

Sewerage.—The sewage originating at Mariemont is discharged into the sewerage system of the City of Cincinnati. As Cincinnati will probably find it necessary to provide sewage treatment in the future, it was advisable to keep the volume discharged into the city system at a minimum and to this end a separate system of sewerage was installed, the storm and surface water being cared for in pipes entirely independent of those carrying domestic sewage. The Mariemont sewerage system serves a total area of about 500 acres, includ-

ing besides Mariemont proper, adjacent territory which drains naturally into Mariemont. The sewerage system empties into a single outfall sewer which, in turn, discharges into one of the main sewers of the Cincinnati sewerage system. Vitrified clay pipe, with overfilled cement joints, has been used for all sewers except in one instance, where a 10-in. cast-iron pipe siphon crosses under a deep gully.

Drainage.—Storm-water drains discharge into natural watercourses at numerous points throughout the development. These drains are of ample capacity to serve both Mariemont and the entire drainage area tributary thereto. By permitting the storm-water drains to discharge at numerous points a considerable saving has been made in the length of large pipes which would have been needed if both domestic sewage and storm water had been cared for by a combined sewerage system. The storm water from the streets enters the drains entirely through street inlets of the gutter-mouth type, no catch-basins being used; access to the inlets is provided by a light-weight frame and cover in the sidewalk. At Mariemont the streets will be kept practically free from dirt and other material which might enter the drains and clog them, and as the drains are built to give good self-cleansing velocities no trouble from this source is anticipated. The use of inlets instead of catch-basins not only reduced construction costs but also eliminated the maintenance cost which would have been necessary for cleaning catch-basins. Vitrified clay pipe is used for the smaller sizes of drains and reinforced concrete pipe for the larger drains.

Central-Station Heating.—One of the distinctive features of Mariemont is the central-station heating system. A boiler-house of fire-proof construction, shown in Fig. 7, is located under the bluff at the southeasterly corner of the site adjoining the tracks of the Pennsylvania Railroad and at a level about 100 ft. below that of the main portion of the town. Its equipment includes three Heine water-tube boilers (with provision for a fourth); each boiler has a normal rating of 379 h.p., and is to be operated at 150 to 200% of rating; also, automatic stokers, overhead bunkers, coal conveyors, and other modern devices for the economical production of steam.

At the boiler-house steam is delivered to the main pipes of the distribution system at a pressure of 50 lb. per sq. in. and is available for private consumers throughout the larger portion of the town, as well as for the heating of public buildings. By the use of reducing valves at the individual buildings steam is supplied at low pressure to consumers, the amount being determined by individual meters which measure the condensate. The steam-heating distribution system is along the rear lot lines instead of in the streets.

Work of Public Service Corporations.—A distinctive feature of Mariemont lies in the fact that all utilities are underground. This accomplishment was simplified by the fact that those in charge of the development of the town had to deal with only two public utility corporations, namely, the Union Gas and Electric Company and the Cincinnati and Suburban Bell Telephone Company, the officers of which entered heartily into the spirit of "the Mariemont ideal". As soon as the streets were graded these companies installed

pipes and conduits, with the result that everywhere throughout Mariemont there is "service", and has been practically from the beginning. This, with the early installation of water mains and sewers, enabled the construction of the town to proceed rapidly. Fig. 8 shows the relative positions of public utilities in a typical 50-ft. street.

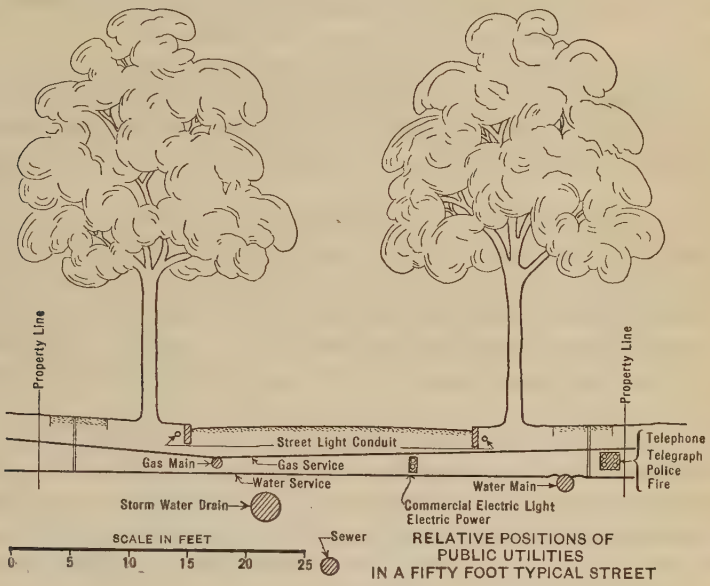

FIG. 8.

Electricity.—Electric current for both domestic use and street lighting is delivered over a high-tension line to a transformer station, a building expressly erected for this purpose near the northerly edge of the town. This building, of brick and concrete, contains the most efficient equipment and is automatically operated from the electric company's main power plants. Leading from this sub-station are two circuits, lighting and power, and at short intervals along these lines there are transformers which reduce the voltage for domestic use to 110 (alternating current). The entire distribution is carried by underground conduits which lie in the street close to the curb line.

Street Lighting.—The street lighting system of Mariemont has been most carefully planned, the town being zoned for lighting purposes according to best modern practice. The town center will be intensely lighted from twin light standards, 18 ft. high, each unit having one 1 000 c-p. incandescent lamp. These standards, which here are 100 ft. apart and opposite each other, are of cast iron, simple and dignified in design. The same model of standard is used throughout the village, although the type of lamp termination differs according to locality. Main thoroughfares and boulevards are lighted by single-light 600 c-p. units, 15 ft. high, and spaced from 100 to 150 ft. apart in staggered formation. Residential streets have 400 c-p. units which are 12 ft. high and approximately 175 ft. apart. After close study of existing

street lighting systems, the officials of Mariemont determined on the use of the Novalux Form 12 lighting unit, with alabaster rippled globes and canopies, for streets and boulevards. Form 25-A Novalux units, each equipped with a 250 c-p. lamp and dome reflector, provide lighting for the service lanes in the rear of the houses. Current for street lighting is distributed by lead-covered cable sunk in the ground only a few inches, thus following the latest urban method.

Gas.—Gas for Mariemont is brought to the town limits through a high-pressure supply main connecting with the Cincinnati system. The pressure is reduced to normal domestic pressure by a standard high-pressure regulator. The distribution system is built as a gridiron. Service pipes are carried to every lot.

Telephone and Telegraph.—To insure absolute absence of overhead wires, the Telephone Company has installed a complete distributing system of underground conduits in the streets. By this means all telephone and telegraph lines, including trunk lines from Cincinnati to the east, are placed underground. Service connections have been brought to every lot. It is anticipated with the rapid development of the radio there will be no need for aerials.

Street Construction.—Street construction has been carried out in a way to promote both usefulness and economy. As soon as the streets were brought to sub-grade, all underground structures within them were placed, whereupon a temporary surfacing of gravel was laid on the sub-base, this surfacing being adequate to permit the use of streets during building operations and later serving also as a foundation for permanent pavements. The permanent street surfacing is not laid until after the completion of all underground utilities with their service connections to lots, and generally until the buildings fronting on the street have been constructed. Thus, the permanent surfacing, once in place, need not be disturbed, neither is it subjected to the wear and tear consequent on the use of the street by trucks and other heavy vehicles while construction is in progress. Concrete and asphalt roadway surfacings, with curb-stones and cement sidewalks, are used for the more important thoroughfares about the town center. Bituminous macadam and, in certain cases, water-bound macadam surfacings are used for the roadways of the less used residential streets. Service lanes in the rear of buildings are surfaced with concrete.

Project Organization

The development of Mariemont is being carried out by The Mariemont Company, organized by Mrs. Emery and incorporated under the laws of Ohio. The President of the Company is Mrs. Emery's business representative, Mr. Charles J. Livingood. It required several years to obtain the various parcels of property comprising the site before active planning of the development was begun. The town planner, who also handled the landscape architectural

features of the development, the several architects, and the writer's firm which was retained for the engineering features of the project, have worked in co-operation for the success of the undertaking which is now being built under engineering direction and supervision, a partner of the engineering firm serving as General Manager for The Mariemont Company.

Municipal Operation and Management

At present (1926), one of the principal sections of the town, the northwesterly section known as Dale Park, is completed and its buildings are occupied by more than three hundred families. To care for the existing population, The Mariemont Company maintains the nucleus of a town organization, including a teaching staff for the school, police force, fire department, and community nurse, together with a force for municipal maintenance work.

In the beginning, all the homes constructed by The Mariemont Company are being rented. Ultimately most of them will be sold, and the town will then be placed in the hands of those who live there to control its destiny.

It is planned to incorporate the Village of Mariemont, with its own town government, a mayor and a council, on the city manager plan. The charter will be based on suggestions and safeguards developed through special studies made by the Rockefeller Bureau of Municipal Research. When the town government is established and in full running order, The Mariemont Company will withdraw. Community buildings, parks, playgrounds, streets, and other features of public nature, will then be presented by Mrs. Emery as her gift to the new town of Mariemont.

DISCUSSION

CHARLES WELLFORD LEAVITT,* M. AM. SOC. C. E. (by letter).—Inasmuch as Mariemont has been typified in Mr. Fay's paper as a "practical example of the application of the principles of community development" and as being "unique in the history of town planning in the United States" and is called by the author, "a National exemplar", it would seem as if a little more should be known in detail about certain statements in the paper which are of great interest to town planners, particularly if Mariemont is to be admitted as the standard to follow in planning residential communities.

Is it possible to set out a complete National example in community building? Each problem presents separate and independent conditions. Topographically, Mariemont lends itself to a maximum of principles governing city planning and the planners themselves have been so well grounded in their profession as to be able to make full use of all possibilities.

The general principles of design as expressed in the paper impress one as sound. Many if not all of them have been used elsewhere and satisfactorily, although the writer knows of no instance where they have all been used together in harmony as at Mariemont. In bringing all the most improved methods of design and execution into co-operation as has been done in this case so successfully, great credit must be given to the planners who have thus made a distinct advance in town planning.

It might be of great interest to enlarge upon the statement that "it does not provide homes for the very poor; experience thus far has shown the practical impossibility of providing new homes for people of the lowest economic scale". It is also stated that "Mariemont is intended primarily as a place of residence for families of widely different economic standing, and especially wage earners". Is it to be understood from these statements that Mariemont only accommodates the well-to-do and those of moderate means? Some one must perform duties which return small pay (though a living wage) and of these there are many, and they are necessary for the functioning of any community, whether residential or industrial. Where are these people to live if not in Mariemont? To some extent these people, not very poor, but of less than moderate means, have been provided with new houses elsewhere in other new towns. Could it not have been done in Mariemont? Or has it been done? Almshouses are the only examples known to the writer of new structures having been provided for the very poor, those of the lowest economic scale. With this exception the very poor have always been obliged to house themselves, usually in the houses discarded by the well-to-do, who build and live in new homes.

It would be interesting to be informed of the rentals received for the smaller houses, or those "many small flats or apartments, some even a single room", provided to "meet the needs of those who can afford only low rentals". What is the amount of these low rentals?

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The church, the recreation facilities, the housing so carefully planned by various architects are as nearly perfect as modern art can devise them and should give the greatest comfort and satisfaction to the inhabitants of Mariemont.

The street system seems perfect except possibly the direct "major thoroughfare from Cincinnati passing through the town center". It would be interesting to know if this major thoroughfare might not have passed outside the community with a branch into the town center so as to keep out the through traveler, rather than congest the town center with people having no business in Mariemont.

The town plan; the engineering, with the distribution systems for the water, gas, electricity, telegraph, telephone, and police and fire alarm; the collection systems of drains and sewers; and the street pavements seem to be beyond criticism. The installation of service pipes to every lot before the street pavements are laid is especially to be commended.

The central heating plant, the one service which is developed in Mariemont from the beginning to the delivery of steam into the public buildings and homes, is certainly the distinctive engineering feature. Can this be run with such economy as to compete with ordinary methods? If so (and the cost figures on this would be interesting), it will be a godsend to the coal heaver and the ash remover.

While it is stated that Mariemont is not a philanthropy, surely the munificent act of Mrs. Emery in providing the community buildings, such as the church, school, hospital, parks, playgrounds, streets, and other features of a public nature, as "her gift to the new town", relieves the residents of the necessity of financing these facilities which are a necessity in any well-ordered community.

The things that have made for progress, those things that are appreciated best, and from which the greatest amount of good, pleasure, and enjoyment are derived, are those provided by one's own efforts and funds. With this great gift and the fact that the profits from this development "are restricted to a low figure", will not this community be spoiled by not paying for those facilities which other communities do provide, and in which they take pride because they have done so? With so much provided for them will not Mariemont's residents have little left to do but to sit back in luxurious ease and eventually become stagnant and decadent?

It will be of great interest to all town planners to know whether Mariemont develops a soul; and yet how else can such perfection be secured in the development of a residential community which so frequently happens rather than is planned?

Fully realizing the noble motive that prompted Mrs. Emery in founding Mariemont and in making these magnificent gifts for real and lasting service for the better housing of mankind, the writer insists that this is not "a real estate development on normal American lines", but that an atmosphere of charity will always pervade Mariemont.

In order that Mariemont, or any other place which has been brought into existence by the act of an individual, shall have a "soul" and the spirit of co-operation and advancement—assuming that all things are done as Mrs. Emery has performed in Mariemont, so carefully, thoroughly, and generously—it might be necessary to insist that the residents in this town pay for what they get, at market prices, so that patronism does not become evident, and the characters of the individuals will be developed. We must not have indolence, but virile active workers if communities are to keep pace with the general world advancement. The soul of the town and the spirit of freedom and the buoyancy of advancement will be measured by the strength of the people of the community and their activities.

Mariemont may serve a purpose to determine how far modern art and science which here have been so skillfully combined may go and yet develop a spirit to carry on the traditions which have made the towns and cities of the United States what they are.

It would seem that if Mariemont's citizens were to carry the whole financial burden and if accommodations were provided for those of less than moderate means, this very beautiful conception will truly become an "examplar" for all to follow.

Frederic H. Fay,* M. Am. Soc. C. E. (by letter).—Several points have been raised by Mr. Leavitt, some of which it is now possible to answer more definitely in view of the time that has elapsed since the publication of the paper. It must be borne in mind that Mariemont is yet a town in the making. Construction is still in progress, and the town is gradually being peopled as residences and other buildings are completed. Matters of policy concerning the future incorporation of the community are not yet fully determined and are subject to modification to meet conditions that may exist at that time. For the present, the development of the community and the temporary management of its affairs are in the hands of the Mariemont Company.

Mr. Leavitt inquires as to rentals charged for residential properties. In the "group houses" of frame construction, a home of four rooms and bath, equipped with furnace, may be rented at $35 per month. A five-room home with bath, in a four-home "group house" of brick and stone construction, such as is shown in Fig. 4, rents for $45 per month. This latter figure applies also to four-room apartments in duplex houses, such as are shown in Fig. 5, with corresponding rentals for larger apartments. Single houses of six and seven rooms, of stucco construction, shown in Fig. 6, rent from $70 to $80 per month. A single stone or brick house of seven rooms, located on the central heating lines, together with a garage, rents for $100 per month and upward. Homes less expensive than the cheapest noted here are still to be provided in the "Westover" Section of Mariemont which lies at the westerly end of the development, and is separated from the central part by a ravine. This natural barrier is being developed for park purposes. The Westover Section, which is immediately to the left of the section shown in Fig. 1, has direct rail connection and is to be developed, in part, for selected industries,

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and, in part, for homes for workers in these industries, as well as for those employed in Cincinnati.

In regard to the cost of heating houses from the central heating plant, it is yet too early to obtain accurate figures as the plant thus far has not been operated at full capacity. To the present, the Mariemont Company has made an arbitrary charge of $20 per month, during the heating season, or six or seven months, for heating an average size six-room house. It is planned, however, shortly to establish a charge approximating $1.10 per 1 000 lb. of steam as measured by a condensate meter. If the tenants or home owners do not waste heat, this charge will amount approximately to the $20 per month heretofore paid by the occupant of the average six-room house.

In commenting on the street system, Mr. Leavitt inquires if the "major thoroughfare from Cincinnati passing through the town center" might not have been located outside the community with a branch to the town center, in order to keep out the through traveler and avoid congestion. That would hardly have been possible since Wooster Pike, the thoroughfare referred to, is a long-established and important State road, and the location of the town center was practically fixed by the conditions of the site. A by-pass road does exist, however, as noted in the paper (page 741), a boulevard along the edge of the bluff, overlooking the Little Miami River, that extends the entire length of the property, a distance of 1½ miles. This boulevard will eventually be linked with the Boulevard System of Cincinnati. It takes through travel entirely to one side of the main development and brings traffic past the Concourse from which an attractive view may be had of the valleys of the Little Miami and the Ohio Rivers. Furthermore, another street through the central part of the development, about midway between Wooster Pike and the Boulevard along the bluff, will also serve as a by-pass route avoiding the town center.

Mr. Leavitt inquires whether Mariemont will develop a "soul" in view of the projected gift by Mrs. Emery to the town of many features of a public nature. He suggests that to secure a spirit of co-operation and advancement, residents of the town might well be required to "pay for what they get, at market prices, so that patronism does not become evident, and the characters of the individuals will be developed". As against this it may be suggested that in communities which develop slowly, unlike Mariemont, public improvements are not provided all at one time, and the entire cost is not borne by a single generation. Residents of such communities profit from the advantages provided by those who preceded them, often with little appreciation of what has been done by others for their comfort and well-being. Were the people of Mariemont to assume the entire cost of the park and recreational features, which for all time will serve not only that limited community, but also residents of a far wider territory, it would put on them a burden which perhaps would be prohibitive. On the other hand, "patronism" does not exist to the degree that Mr. Leavitt may have in mind.

Recently, there has been created a trust known as the Thomas J. Emery Memorial, which has taken title to the parks, playgrounds, and recreational

facilities of the community that are dedicated forever to public use. These, like the hospital, will be maintained as a memorial to Mr. Emery. Certain of the public buildings, like the community church, the municipal museum preserved in the oldest house in this section, and also the projected community building, will be Mrs. Emery's gift to the new town. Likewise, the land required for streets has been dedicated to public use. The costs of street surfacing, and of municipal utilities, such as the water, sewerage, and drainage systems and underground conduits, have been apportioned upon, and are included in, the lot prices. It is now intended that when the town becomes incorporated it shall take over and pay for certain buildings, such as the school and fire station, and that it will itself provide additional facilities of like character when they are needed.

Although the town is in the making and the population is moving in as rapidly as homes are made ready, there is being developed among the people who are already there a real community spirit and pride in the new town. The importance of this psychological factor has not been ignored by those who are responsible for the development. One who breathes the atmosphere of Mariemont, and sees the spirit already prevalent among the residents who have come there thus far, feels assured that the new Town of Mariemont will have a soul.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1645

THE CINCINNATI CITY PLAN IS NOW LAW*

By George B. Ford,† Esq.

With Discussion by Messrs. Alfred Bettman, Edward M. Bassett, Arthur C. Comey, E. P. Goodrich, Thomas T. Towles, Harland Bartholomew, and Thomas H. Reed.

For the first time in the United States, a complete, comprehensive city plan has become the law of a city. The whole plan actually can be enforced, thanks to the exceptionally broad powers granted to the City Planning Commission of Cincinnati, Ohio, by the State laws and the City Charter.

Elsewhere throughout the country, city planning commissions are strictly advisory. They rarely have any power except that in some States they have the right to control the layout of subdivision plats and in some they have art jury powers permitting them to veto the location of public works of art and sometimes to control the appearance of public buildings and structures. The authority of the City Plan Commission of Cincinnati goes infinitely further than this, for under the statute and the charter, there can be no departure from any item of the city plan, once adopted by the Commission, except by a two-thirds vote of the full membership of the City Council, after public notice and hearing, accompanied by the approval of the department head most affected.

Therefore, the problem which deserves most earnest discussion, is whether city planning commissions throughout the country, and regional planning commissions as well, shall continue to have merely advisory powers or whether extensive powers, such as those granted by the Ohio statute, are really practicable or desirable.

In Massachusetts, for example, the planning boards have no power whatsoever except that their existence is compulsory. The argument for strictly

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

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advisory powers is that if the city planning board cannot convince the public and the city officials that its ideas are the best, there must be something wrong with the ideas. The further argument is made that legislative authority should not be divided, but rather concentrated in one body, that is, the city council, which alone is strictly responsible to the electorate. The claim is made that the State Legislature has no right to delegate to a non-elective, non-responsible body any legislative power such as making the city plan the law of the city. In any case, no city council would consent even to sharing the control of the city plan with another body.

In Ohio, however, the claim is made that city planning is a highly specialized and a highly technical matter; that the preparation of a plan that is worth anything, requires the concentrated effort of a selected group of exceptionally intelligent and experienced citizens, aided by the best technical advice. It is maintained that the average city council has neither the time nor the experience necessary to frame an effective city plan, and, in fact, is only too glad to shift the burden, and the inevitable charges of favoritism involved in most planning matters, to another body. It is felt that the making and the promulgating of the city plan is comparable to the making and issuing of health regulations by a health board, or to police or traffic regulations by a police board, or fire prevention regulations by a fire commission. In each of these cases there appears to be little question but that the State Legislature has the right to delegate specific legislative power to these strictly appointive boards. It is merely an extension of the same idea to delegate to a city planning commission the right to frame and enact a city plan, of course, giving the city council a veto power over it, provided this veto cannot be exercised too easily.

The City Planning Commission in Cincinnati consists of seven members with the Mayor as Chairman and with the Director of Public Service and the President of the Park Board as *de facto* members. The Commission contains in its membership, three manufacturers, two lawyers, and one physician of the community.

The power and duties of the City Planning Commission according to the statute are as follows:

"Sec. 4366-2. The powers and duties of the commission shall be to make plans and maps of the whole or any portion of such municipality, and of any land outside of the municipality, which in the opinion of the commission bears relation to the planning of the municipality, and to make changes in such plans or maps when it deems same advisable. Such maps or plans shall show the commission's recommendations for new streets, alleys, ways, viaducts, bridges, subways, parkways, parks, playgrounds, or any other public grounds or public improvements; and the removal, relocation, widening or extension of such public works then existing. With a view to the systematic planning of the municipalities, the commission may take recommendations to the mayor, council and department heads concerning the location of streets, transportation and communication facilities, public buildings and grounds. The commission shall have the power to control, preserve and care for historical land marks; to control in the manner provided by ordinance the design and location of statuary and other works of art, which are or may become the property of the municipality; and the removal, relocation and alteration of any such works belonging to the municipality; and the design of harbors, bridges, via-

ducts, street fixtures and other public structures and appurtenances. Whenever the commission shall have made a plan of the municipality, or any portion thereof, no public building, street, boulevard, parkway, park, playground, public ground, canal, river front, harbor, dock, wharf, bridge, viaduct, tunnel, utility (whether publicly or privately owned) or part thereof shall be constructed or authorized to be constructed in the municipality or said planned portion of the municipality, until and unless the location thereof shall be approved by the commission; provided that in case of disapproval, the commission shall communicate its reasons for disapproval to council, and the department head of the department which has control of the construction of the proposed improvement or utility; and council, by a vote of not less than two-thirds of its members, and such department head shall together have the power to overrule such disapproval. The narrowing, ornamentation, vacation or change in the use of streets and other public ways, grounds and places shall be subject to similar approval, and disapproval may be similarly overruled. The commission may make recommendations to any public authorities or to any corporations or individuals in such municipality or the territory contiguous thereto, concerning the location of any buildings, structures or works to be erected or constructed by them.

"Sec. 4366-3. The municipal planning commission shall be the platting commission of the municipality, and all the powers and duties provided by law for platting commissioner or commissioners of municipalities shall upon the appointment of a municipal planning commission under this act, be deemed transferred to such commission.

* * * * * * * *

"Sec. 4366-7. The city planning commission of any municipality shall have the power to frame and adopt a plan or plans for dividing the municipality or any portion thereof into zones or districts, representing the recommendations of the commission, in the interest of the public health, safety, convenience, comfort, prosperity or general welfare, for the limitation and regulation of the height, the bulk and location (including percentage of lot occupancy, set-back building lines, and area and dimensions of yards, courts, and other open space), and the uses of buildings and other structures and of premises in such zones or districts."

The powers and duties of the City Planning Commission according to the City Charter of 1918 are almost identical. The City Charter even agrees with the State law in requiring all sub-division plats not only within the city but for three miles outside it to be approved by the Commission before they can be offered for record and before the streets can be dedicated.

Since the City Plan of Cincinnati has been law every question that has arisen as affected by the City Plan, has been decided in accordance with the plan. All city departments are co-operating enthusiastically. In no instance has the City Council over-ruled the City Plan. It is true that several minor modifications have been made in the plan as a result of further study by various city departments. In particular, a more temporary, but more immediate solution of the traffic difficulty at Brighton Corner was adopted by the City with the sanction of the Planning Commission.

During the short period the plan has been operative, a fire-engine house and several schools have been located in accordance with it. The site proposed for the Public Auditorium has been retained for that purpose despite strong demands that it be released for other uses. The Workhouse property has been retained as a playfield site despite demands to the contrary. The

traffic and transit difficulties at Peebles Corner are being solved in compliance with the plan. The various down-town roadways are being widened according to the plan despite strong opposition. The street-paving program as well as the Eighth Street Viaduct reconstruction also accords with the plan. The location of a railroad union passenger station and rights-of-way relocation are being carried out according to the plan. New parks and playfields conform with the plan. All the sub-division plats, even for three miles outside the city limits, accord with the plan. Traffic regulations are in accordance with the plan. In general, it may be said that there has been no departure from the plan except in minor instances where the City Planning Commission was convinced that it was possible to improve on it.

The general impression of those who are watching the effect of the Cincinnati method is that it is proving highly successful and is a distinct improvement on the strictly advisory powers of most other planning commissions. It means that the presumption is in favor of the plan because it is the law and also because it was worked out with a great deal of care and thought. It means that the obligation rests with any one who disagrees with the ideas presented to prove that, all things considered, the plan can be improved upon and then the burden rests on him to present a better solution and to convince the City Planning Commission or the City Council that he is right. In other words, it means stability to the plan as a whole, features which do not exist in the same degree in most other States.

It is true that in a city such as Memphis, Tenn., for example, the Mayor and the City Council, on account of their interest in the plan, may virtually give it the force of law that approximates the effectiveness of the Cincinnati plan. This, however, presupposes a strong and continuing interest in the plan on the part of the leading city officials.

In Springfield, Mass., the City Council passed an ordinance that no city official or department should depart in any way from the city plan without first trying to obtain the approval of the City Planning Board, and failing that, the approval of the City Council. This method also has worked well in practice, thanks to the activity of the City Planning Board and the general public interest, although, again, the effectiveness of this method depends on a continuing active interest in the plan on the part of leading public officials.

The Cincinnati method has the advantage of continuing its effectiveness through changing administrations and even over periods of possible public apathy.

The writer had felt that the powers of a city planning commission should be purely advisory, but in light of the experience in Cincinnati, he is now convinced that if city planning is to be vital in the functioning of communities, the Cincinnati method should be applied generally to city regional planning. Of course, the success of this method depends on the quality of the planning commission, but the writer's experience in working with more than one hundred different commissions has convinced him that most of the members would measure up to the job. In fact, the very seriousness of the responsibility entrusted to such commissions would inspire them to make the plan a masterpiece in which all would take pride.

DISCUSSION

ALFRED BETTMAN,* ESQ.—Probably all are agreed that the city plan, however comprehensive, should have a considerable influence, even a considerable degree of control, in carrying itself out; that is, in actually becoming the design which is executed in the process of developing the city.

The problem of how to get these effects is not one of engineering in the technical sense, nor of city planning, nor of law. It is really a problem of political science, and the Ohio statute, referred to in the paper, is the product of applying experience in practical politics to the problem of city planning and does not represent any fine-spun theory.

The basis of that statute is as follows: The regular officials of the City Administration as, for instance, the City Engineer, and the members of the legislative body, the City Councilmen, have an enormous mass of ordinary, every-day, pressing, urgent, routine problems. They are subjected to the political currents of the day—"political" not in any derogatory or sinister sense, but in the sense of vocal popular opinion or pressure of the day. Thus, the daily problems of the expenditure of public funds and the selections of public improvements take considerable of their time and energy and tend to be decided from the point of view of day-by-day pressures and day-by-day urgencies. The officials who have these daily problems are, therefore, by virtue of that fact, not the men in whom to repose the making of the city plan, which is a design for a long time in the future. Such a design therefore needs, as a matter of human nature, to be made by some body aloof from these daily urgencies which make it impossible to think in terms of long periods.

Consequently, a City Planning Commission should be created, with representation from the regular City Administration, so that there may be coordination, but primarily aloof from it.

How shall the work of the Planning Commission, assuming it to be well done, be placed in a legal position actually to impress itself in results throughout the period of the plan? In order to think this problem out clearly, one must distinguish between that part of the plan which relates itself to the control of private property—that is, developments on private property, known as zoning—and that part which relates itself to the grand or master design for the location of permanent public improvements, such as streets, boulevards, markets, utilities, etc.

In so far as the plan is related to the control of the private owner's use of his own property, it obviously must be converted into law, and while naturally agreeing with Mr. Ford's praise of the Ohio System, the speaker has to disagree with his terminology. By "law", if it be used in any accurate sense, is meant a rule of conduct behind which there is a sanction enforced by the Courts. Therefore, this plan, which is a map and explanatory text, does not become law, as far as it controls the private use of the owner's property, until there is placed behind it some sanction enforced by the Courts.

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That process of making law should not be reposed in any appointive administrative body, because it is of the very essence of the American constitutional system that law-making be reposed solely and exclusively in elected representatives of the people. Therefore, the converting of the zoning plan into law should be, and, in the Ohio statute, is, reposed in the City Council to be exercised by means of the passage of a zoning ordinance.

What effect should the Planning Commission's technically expert plan have on the text and the map of the zoning ordinance? The Ohio statute states that before the Council may depart from the zoning plan as made by the Planning Commission, it shall refer the departure back to the Commission, and if the Planning Commission does not agree to amend the plan, then a two-thirds vote of the Council is required to override the Planning Commission's opinion. That is something that is more than advisory and less than compulsory. It is a midway position. It does not differ, however, from many other examples in the statutes of every State in which the Council must act by a two-thirds vote. It is quite customary, for instance, to require a two-thirds or three-fourths vote of the membership of the legislative body to override the veto of the mayor, or governor, or president. Similarly, if the Planning Commission vetoes a departure from the plan, then approval by two-thirds of the Council, instead of the ordinary majority, is necessary to convert that plan into the sanctions and the methods of enforcement which are characteristic of law.

How about that part of the plan which relates to public improvements—where the future streets shall be; where the boulevard system shall be extended; where the railroad terminals shall be; and all the other features of this grand master design which are to be carried into effect throughout the years of a generation or more? This is a point not raised by Mr. Ford, but it is a point which is involved in the process of drafting a city planning statute. It seems quite obvious that the City Council of any particular day or month or year should not be given the power to approve or disapprove the whole initial master design or city plan, because, by virtue of the method of choosing members of the Council, its procedure, the technical facilities at its disposal, and its subjection to all the current pressures of the day, it has not the qualifications, the equipment, or the motives which are appropriate to making a grand design of future public improvements running through a long period of years. Therefore, the making of such a design cannot appropriately be left to a City Council, but needs to be in this more aloof type of body known as the City Planning Commission, assisted by technical skill.

What effect, however, shall this grand design have on the actual location of future public improvements? From year to year a City Council has the problem of raising sufficient monies and expending them for the things that are to be built that year. In other words, it appropriates the money, whether it be tax funds, bond funds, or other funds. Whichever type of fund may be expended, it is again fundamental in the American political system that in the elected representatives of the people con-

stituting the legislative body shall the power of the actual expenditure of public money be reposed, and that there ought to be no power in any planning commission, or any administrative board, or any appointed officials to decide finally the expenditure of any particular sum of money on any particular type of improvement at any particular time; and the Ohio statute does not attempt to give any such power to the Planning Commission. All that it attempts to do in the case of future public improvements is exactly that which the speaker has attempted to describe in relation to the zoning ordinance—it is something more than merely advisory and something less than compulsory, namely, that before Council may locate, by its own expenditure of public monies, or by a franchise, a public improvement or utility other than at the place designated in the plan, it shall first have to refer this proposed departure from the plan to the Planning Commission, and if it does not succeed in convincing the Commission to amend the plan accordingly, then a two-thirds vote of the Council is requisite to override the plan and to locate the improvement of the utility in violation of it.

That is quite different, at least in the terminology of lawyers, from making the city plan law. That is, however, it seems to the speaker, giving a sensible amount of force to the city plan, to assure that the plan cannot be carelessly overridden. The plan may not be simply ignored or forgotten. Council must take it into account to the extent of having the fact that the plan is there called to its attention, to the extent of having to go to the Planning Commission with any proposal that violates or departs from the plan, and to the extent that more than a bare majority of the Council is needed to depart from the plan. This is simply keeping the plan alive, and with some influence, in the ordinary routine process of legislating from day to day in the expenditure of public funds. That is the theory of the Ohio legislation—not theoretical in any sense which is contrasted with the practical, but the theory of the applied politics of it.

The Cincinnati Plan was officially adopted in May, 1925. It is a little soon to draw any firm conclusions as to how it will turn out, but it looks as if the theory will succeed and as if the plan will be given some strength, more than a mere negligible piece of advice.

For instance, shortly after the plan was adopted, the City Engineer sent to the Planning Commission certain detailed plans and specifications. These called for the re-surfacing of certain streets at their then width, whereas the City Plan called for a widening of these roadways with a corresponding narrowing of the sidewalks. The streets were in bad condition; there was tremendous pressure to have them re-surfaced immediately. The narrowing of the sidewalks would deprive the property owners of the use they were making of that space. They had machinery beneath the sidewalks and areaways, and entrances to basement restaurants, etc. Therefore, the City Plan disturbed their free use of city property and they opposed it in that respect. Under the urgency of re-surfacing and the private free use that was being made of the sidewalk space, Council passed the ordinances in accordance with the plans and specifications without paying any attention

to the general plan. This was not called to its attention before it passed these ordinances. Thereupon, the City Solicitor notified Council that the ordinances could not take effect, and that he would enjoin the re-surfacing of the street at the original width unless the ordinances were first submitted to the Planning Commission.

This power given to the plan, something more than advisory, forced a reconsideration. The Planning Commission took the matter under consideration, and the private property owners subjected the Commission to the same pressure. The Planning Commission, however, was not as amenable to that kind of pressure as the Council and refused to depart from the plan, and that is where the matter now (1926) stands—the Planning Commission refuses to depart from the plan and the plan has sufficient popular prestige so that Council does not feel like overriding the Planning Commission.

The city owns a piece of property that was the former site of a hospital. There has been a movement on foot to sell it as it is very valuable. The plan designates this site as the location of the future Public Auditorium. Thus far, the plan is winning; and it looks as if it will win in the end.

Similarly, there is a movement, on the part of the Cincinnati Baseball Association, to acquire another piece of property owned by the city for a new baseball field, which piece of property the plan designates as the location of the public City Athletic Field. The Baseball Association finds itself under the necessity not merely of creating a favorable sentiment in Council—and it would be very easy to do that because baseball is the most popular sport in the city—but also of convincing the Planning Commission. At least the City Council will not act until the Planning Commission has had a good opportunity either to amend the plan or stick by it in that particular.

It may be seen, therefore, that ease of departure from the plan is much lessened by giving it the legal status conferred by the Ohio statute which Mr. Ford has described.

Edward M. Bassett,* Esq.—The speaker has given considerable study to this very interesting paper of Mr. Ford. His point of view is almost the same as those of the other discussors, the only difference being as to where to place the emphasis. One of the discussors plainly would put the emphasis on public opinion, and he may be right; but the powers of the community have not been crystallized by public opinion so as to bring about better town planning. The speaker would probably give emphasis to this crystallization of community powers into laws that will bring results.

Officials and citizens of outside towns ask, "How can we prevent the issue of building permits on misplaced private streets?" The answer is, "You cannot prevent it. There is no statute in the State of New York under which you can prevent it, and no ordinance of your village under which you can prevent it." They ask, "How can we prevent a lane being laid out that upsets the plan of the village?" and they can hardly believe it when they are told that they cannot prevent it, that "you can spend some money if you want for

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fighting in Court, but you are going to be beaten because the community power in that respect has not been put where you can avail yourself of it."

Many have become convinced that a planning commission must be more than merely advisory. It is now well recognized that a State Legislature can provide for the appointment of a body to administer a subject. The legislative body prescribes the rule of conduct or the limits within which that administrative body may act on details. The advantage in this method is that although a legislative act cannot be modified by the Court, the doings of an administrative body, acting under a rule duly prescribed, can be reviewed by the Court and, if necessary, sent back to be done again. Therefore, there is great elasticity and adaptability in a functioning administrative body such as many believe planning commissions can be.

Two of the most highly organized laws for city planning in the United States are those of Cincinnati, Ohio, and New York, N. Y. The City of New York is not always considered as highly organized in its course of action; but through a long period of years there has been built up a wonderfully good procedure. In some respects, the City of Cincinnati has improved on it.

The speaker would like to compare some of these improvements by a sort of parallel column method. New York City has an official city map or plan. It covers, however, only streets and parks. It will be noted that it covers only those land areas which are the subject of dedication. The first step in a taking is the placing of an area for a street, open place, or park on the official city map. The next step is the draft damage map, which finds its data on the official city map. Consequently, the official city map must be precise.

In Cincinnati, the official plan of the city shows streets, parks, and open places, but it also shows sites for public buildings, locations of public utilities, and all kinds of city planning projects. The real difference is that Cincinnati includes in its official plan those things which are not the subject of dedication. In that particular respect there is a permanency, or there ought to be a permanency, in the New York official map which one can hardly expect if the same control applies to public building sites and public utilities. Changes of mapped streets and parks should be made more difficult of accomplishment than changes of grades, locations of proposed public utilities, or proposed sites for public buildings.

Next, New York City has no planning commission, but every amendment to the official city map is reported on by the Chief Engineer of the Board of Estimate and Apportionment. Therefore, it is the legislative authority advised by the Chief Engineer that makes changes of the official city map possible. The City of Cincinnati, however, has a planning commission which is assisted by the City Engineer and its decisions should carry a great deal of weight. The reports of the late Nelson P. Lewis, M. Am. Soc. C. E., and, later, of Arthur S. Tuttle, M. Am. Soc. C. E., the only Chief Engineers of the Board of Estimate and Apportionment of Greater New York since 1904, have been as sound and disinterested as those of any commission could be.

Next, in New York, plats must be approved by the Board of Estimate and Apportionment, or by the Chief Engineer before they can be placed on record.

In Cincinnati, they must be approved by the Commission before they can be put on record.

Then, in New York, under State law, it is a misdemeanor to make a deed of land before the plat is filed, if it is land in a sub-division. This is also true of the Cincinnati law. In New York, the law is a dead letter and will probably turn out to be the same in Cincinnati.

There are certain shortcomings in these two plans, however, and the speaker will use both cities to show how wide these shortcomings are. In either city private streets can be laid out by any citizen who chooses to do so on his own land, without any power on the part of the city officials to stop it. They can refuse to make it a public street, but the man who owns his own land can lay out a private street. Next, the man who lays out those private streets can ask for permits to put up buildings on them and the authorities in either New York or Cincinnati cannot lawfully refuse the building permit. A man, in either city, may get a permit on a private street, which may be entirely in conflict with the city plan, and when twenty or thirty houses have been built on a private street, it is only one step to persuade the authorities to put in the sewer, lighting, etc. They cannot very well tear down the houses, and they make the best of a bad situation. The law in all cities is defective in that it allows private streets to be laid out contrary to the plan and permits houses to be built as a matter of legal right on any misplaced private street.

Next, in every State, except Pennsylvania, as far as the speaker knows, an applicant for a permit to construct a building within the limits of a mapped street not yet opened, can obtain it.

Again, small parks, especially small parks for playgrounds, are to-day just about as much of a need in a big modern city as a street. Sooner or later the taxpayers on the surrounding land have to pay for them. Sometimes buildings are torn down and the entire cost of land and buildings is assessed. There should be a method of obtaining small parks for playgrounds as automatically as streets. This should be before the land is built over. One is nearly as important as the other, but because the community can exist without playgrounds and it cannot exist without streets, new streets are required, but playgrounds can go until the land is built up and then the houses must be pulled down at great expense to create the playground.

There ought to be, in both New York and Cincinnati, a more intimate method of co-operation between the land owners and the officials in laying out secondary streets. Many mistakes have been made in New York because there is a hard and fast official city plan before the streets are opened, to which the land owners must conform. Sometimes, bluffs have had to be torn down, which had better been left standing. Sometimes, there has not been any conformity with the plainest needs of the environment. Yet the gridiron plan has been so inflexible that there was nothing to do but adhere to it. There are drawbacks to having a fixed plan to which the private owner must absolutely conform. He ought to have a chance, at least, to talk it over with an advisory administrative planning commission.

There ought to be an intimate method of co-operation between the property owner and the officials in the application of zoning. A small business center is needed, and a buffer district for apartment houses and a place for detached houses outside is often needed. Too great fixity prevents that intimate co-operation between land owners and the officials that ought to exist if a rational city planning method is to be brought about. Planning commissions might properly have power regarding details. These shortcomings in great cities are well worth considering in progressive study of city planning.

Some claim that private land is, and that public land is not, the subject of zoning. By and large this statement carries, but that is not the line of distinction. Zoning relates to buildable land, whether it is private or public. In New York, if the Park Commissioner wants to erect a building in a park, it must conform to the height and area provisions because all the park land is zoned. The streets, of course, are not buildable land, and they cannot be zoned, but a site for a public school or firehouse is zoned, and the public authority must conform to the zoning plan just as much as if the land was privately owned.

Arthur C. Comey,* M. Am. Soc. C. E.—Mr. Ford has given three questions to be answered, each of which relates to at least three distinct aspects of the subject of city planning powers. It seems that the only way to get intelligent answers is to split them up accordingly.

The powers under discussion are exercised over three main categories of property: (1) Private property, which under the plan is to remain private but controlled to a degree by zoning, building laws, and similar applications of the "police power", so-called, for the general good; (2) public property, either already existing or to be acquired substantially at once or later by a single act of acquisition, for streets, parks, public buildings, etc., under the power of eminent domain, with compensation for those damaged; and (3) private property which later is to become public and, therefore, needs to be controlled so as to fit in with the city plan when it ultimately is absorbed, this control being at present, in the United States outside of Pennsylvania at least, largely in the stage of proposals either to restrict under the police power permits for private buildings in the beds of mapped streets and other areas, or to take away by eminent domain the right for damages to such buildings when later taken as part of the public acquisition—the latter method being proposed in Massachusetts† and preferred by the "Hoover" Committee on City Planning of the U. S. Department of Commerce.‡

General theories and "fundamental" principles need not give undue concern as the former must meet the actual needs or fall and the latter are apt to be merely substitutes for thinking. Any of the opposing points of view in regard to the questions under discussion can be supported by a slight and often unconscious twisting of the same words and phrases. For example, one is told that legislative powers should not be delegated from the city

* Cons. City Planning Engr., Cambridge, Mass.

† *Bulletin 16*, Massachusetts Federation of Planning Boards; also, House Bill No. 504, Mass. General Court of 1926.

‡ "A Proposed Standard City Planning Enabling Act."

council and then finds delegation justified by labeling the powers as administrative. Some regard, however, should be paid to practice and its evolution and to the ease of application of proposals. Many an otherwise meritorious plan has failed because it is unnecessarily counter to current habits or because it is too cumbersome or involved in procedure.

The main questions of city planning powers, therefore, as applied to each of the three categories previously listed, are:

I.—Shall the city have city planning powers?

II.—Shall any body other than the city council exercise any control?

III.—If so, shall the city plan commission exercise any control?

In regard to Question I, there seems to be unanimity that considerable city planning powers should exist (1) to zone private property, enforce building laws, etc., as at present effective in most States; (2) to carry out a city plan by acquisition of lands needed for streets, parks, etc., also in large degree now in effect in the United States; and (3) to protect the beds of mapped streets and other areas, as yet only proposed.

In regard to Questions II and III, Mr. Ford urges that the city plan commission should have extensive powers to enforce the city plan it adopts, particularly as regards the second and third categories of property, and adduces valid reasons in support of his position. Yet he, as well as the "Hoover" Committee and others, apparently is satisfied that with regard to the first category the opposite course should be pursued. Zoning is at present practically universally enacted and from time to time amended by the city council, utilizing in so far as it sees fit the advice of the city plan commission or a similar advisory body. This accords with the habit and trend of local government to-day, which seeks to centralize control of the larger matters of vital import in the elected body, the city council, thereby dignifying that body and avoiding conflicts that are inevitable where there are two co-ordinate bodies. It would seem that the same method might work as well for other aspects of the city plan. It has not been proven that it does not and, therefore, a logical first step in the evolution of city planning powers would appear to be to give the city council the power to adopt the city plan, and not delegate these powers to any other body. Where this action proves ineffective local option may well permit the electorate, or the council itself, to delegate powers.

Even if the answer to Question II is in favor of delegation of power, there are arguments against giving it to planning commissions. Such bodies are preferably composed chiefly of high-minded citizens donating their services. They should be free of administrative detail, and moreover the very fact that they are advisory only assures them greater recognition from all quarters. The arrangement in Rochester, N. Y., with a Department of City Planning under a single head, and an advisory commission, appears to be a practical means of securing both the advantages of a disinterested commission and an efficient administrative agency.

In brief, there is no one royal road to successful carrying out of city plans. In Massachusetts, for example, the effort to apply Mr. Ford's proposals to-day would probably wreck the entire city planning structure throughout the State.

Powers to planning boards would be a red rag in the Legislature, as well as in many cities and towns where these new boards, at first viewed with suspicion, have gradually acquired general confidence, based very largely on the fact of their advisory character. These boards meet as a rule once a month. Were their sessions to any large degree taken up by executive detail, such as approving plats, their ability to deal with the larger aspects of the city plan would be apt to be seriously impaired. It may be argued that they should have staffs capable of handling all this detail. In that case the Rochester method would seem preferable, clothing the administrative head actually responsible with the power and leaving the commission as advisory.

In Massachusetts, few people at present would see much need or reason for departing from good and tested practice until it has been tried out as applied to city planning. It has not been tried out yet because thus far the more vital parts of such city planning powers as those under discussion have not been delegated to the municipality at all. There is no adequate power in Massachusetts communities to enforce a city plan. That power might well first be lodged in the legislative body, then, if this is not successful, that, in itself, will be an effective argument for transferring it to an administrative body. That is exactly the proposed procedure in Massachusetts at present in the matter of protecting mapped streets, the third category noted.

Hitherto in Massachusetts it has seemed worth while to try the established practice of having one legislative body handle everything it possibly can. There, as in most States, the city council adopts the zoning law (except in Boston, where zoning and building laws are State acts). The planning board or other advisory agency draws up a zoning plan and transmits it to the council. Sometimes the council tinkers with it, but more often adopts it intact. In practice this procedure seems to work out. The fact that zoning directly controls private property does not appear to be a fundamental reason why the zoning law should be entirely controlled by the council any more than other parts of city planning, also affecting private property.

In conclusion, it seems generally agreed that more powers are needed in practically every State, Pennsylvania possibly excepted, for enforcing a city plan. Whether such powers are exercised by the city council or by an administrative body may well be left to each municipality, either by the electorate accepting this feature of an enabling act or charter provision, or by the council establishing the procedure by ordinance.

The city plan commission may well be charged with the duty of preparing a complete city plan for adoption by the city council. The control of plats and other administrative details may well be lodged in a single-headed city planning department working in conformity to the city plan. As safeguards for comprehensive planning the council's action on all matters affecting the city plan may properly be required to be held up for a reasonable period pending the city plan commission's report thereon. It may even be required that any action other than conformity to the commission's report shall require special publicity and a three-fourths vote of the entire council.

E. P. Goodrich,* M. Am. Soc. C. E.—Based on a rather wide range of personal experience, the speaker takes the liberty of crystallizing the thought on the question herein discussed. In the first place there must be as rigid a condition as is possible, with as much power as the public will permit in the hands of those who are forming and administering the city plan. At the same time the plan must be as elastic as possible. Those two things seem antithetical, but there must be created by some device, some means of adapting a plan to the needs when something new arises, through a process of evolution. In the original grant of power, of course, one cannot go further than public opinion at that time will uphold. Sometimes, it is possible, by looking into the future, to "put something over" on the public. Luckily, the ones who normally are charged with the formulation of plans are usually looking out for the public interests.

At the same time that a plan is being created and carried out, public opinion must be formed. Sometimes that can be done by the city plan commission, or a similar body, but experience shows that the best method is by an independent citizen body or city movement which will keep behind the public officials and force them to create the best possible law or plan at the time and for as far into the future as possible, and then force them to administer it to the best interest of all the community.

This need of an independent citizen body constantly behind the plan, constantly at the elbow of the officials, even if the latter be the highest class of citizenry, is imperative to get the best results. The speaker thinks he is free to quote Mr. Bettman who stated that even in Cincinnati a citizen body will be necessary continuously to see that the city plan is carried through and maintained.

Thomas T. Towles,† M. Am. Soc. C. E.—Although the speaker has been greatly interested in Mr. Ford's stimulating paper he differs very decidedly from his conclusions. There seems to be an erroneous notion that the city plan could at any time be a fixed conception to which the city's growth must adhere. If the people or the authorities of the city have the idea (and at some time they will entertain such a notion) that a formal plan very difficult of change is imposed on them from the outside, so to speak, it would seem to be almost impossible of effective enforcement. Such would be particularly the case with the smaller cities or towns in which the interest and the influence of the individual is more active and direct than is the case in the large centers of population.

The comprehensive plan of Cincinnati involves not only the details of street layout, but also the particular layouts of parks and playgrounds and even the plans for all public utilities (perhaps, however, general in nature in this case). The speaker's experience has been that if the plan went into all details of this nature the authorities of the town would have to ask the city council for a change in it, every month. A few repetitions of this would simply put the plan in discard. A community is subject to constant change

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† Design Engr., Div. of Eng. and Constr., Stone & Webster, Inc., Boston, Mass.

not simply as regards population, but to change and development as regards its ideas, general outlook, and even its necessities. The city plan must be reasonably capable of taking new form, elastic in its nature, and responsive to the needs and changing conceptions of the community.

The necessity of having a body of citizens outside the constituted authorities of the city earnestly devoted to the problems of city planning has also been discussed. The City Plan Commission of Richmond, Va., is composed of five public officials, the mayor, and the directors of the several departments of the city government. This body of officials simply does not act in any sense as a city planning commission. Each official is concerned almost entirely with the routine duties of his office. Furthermore, it is very seldom that a public official conveys to the people the notion of disinterestedness that would be attached to a commission presumably uninfluenced by political considerations.

Sufficient stress has not been given to the importance of public opinion. To obtain a thorough consistent execution of any largely conceived plan, it is necessary to educate public opinion to some conception of the importance of planning for the community as a whole. Furthermore, it seems fundamental that a plan should be, in some real sense at least, a result of the expressed needs and a product of the will of the community.

Harland Bartholomew,* M. Am. Soc. C. E.—The speaker has been greatly interested in Mr. Bettman's discussion. There are a number of reasons or arguments that might be set up on either side of this question of placing full or partial responsibility in the hands of the planning commission. There is the fundamental objection referred to by Mr. Bettman, namely, the placing of authority in the hands of a non-elective body. Perhaps in law that is the controlling fundamental consideration. Against the argument of making the plan commission purely advisory might be cited the fact that if left wholly and exclusively within the hands of elective and appointive officials who are not members of the planning commission, the fundamentals of the plan are ignored, unappreciated, and successive officials are not bound to consider them; and, what is more, those elective and appointive officials who change office, of course, invariably like to express their individuality to make a record, as it were, for their particular administration in the particular office which they may occupy. The speaker does not believe there is any uniform panacea that can be advanced, or any method than can be offered, for the handling of this question.

The Ohio law to which Mr. Bettman refers would not apply in St. Louis, Mo., for instance, because in St. Louis the very form of municipal government invariably makes the entire Council, Mayor, and full administration of one political faith, and they have acted in unison and as a unit on all matters with only the slightest exception since the adoption of that charter in 1914. Of the votes of Council 99% is always all "yea" or all "nay". There is no such thing as even a minority representation or difference of opinion. From which it may be deduced that the matter of the enforce-

* (Harland Bartholomew and Associates), St. Louis, Mo.

ment of the plan is largely one that has to be studied very carefully in every community. It is not one that can be set up arbitrarily to apply uniformly to all cities.

The speaker would also offer the opinion that the execution of the plan is primarily a co-operative enterprise and not one that must or will lead to checks or differences of opinion. It ought to work smoothly and these halts, discussions, and issues ought not to arise if possible. He will make a prediction with respect to Cincinnati, which is offered purely for purposes of discussion. Mr. Bettman cited three instances where projects have been held up. Within the course of a few years, if many more such instances arise, the Council will become resentful and enough groups about the city will be antagonized to such an extent that there will be sufficient sentiment to cause the Council to repeal its action adopting the plan. Perhaps that might be overcome by a referendum, but even in that case, by the same token, a referendum, sooner or later, might be upset.

This brings the present reasoning on this matter to this: That back of all successful work in government—perhaps "political science" is the term—there must be understanding and co-operation. This, in turn, goes back to public opinion. Despite the fact that a plan commission may get weary of the tremendous effort of keeping public opinion up to the pitch of supporting the city plan, fundamentally that is the proper way to secure the continuous execution of a large proportion of the plan. City planning is no different from any other form of public work. It must be carefully watched and supported by an enlightened citizenship if it is not to fail completely or fall into corruption.

The planning commission has two fundamental jobs. One is to study, and study, and study, indefinitely, always perfecting and developing the plan; and the other is to develop public opinion to the point where that plan will be carried out whole-heartedly and intelligently.

With respect to the first of these, if the planning commission is properly constituted the public officials who have charge of the execution of the plan are going to see to it that the major part of that plan will be intelligently executed. For there is certainly no force in public life and public work greater than that of public opinion, which public officials generally wish to respect.

The speaker would like to offer one or two examples, which he has often cited in discussions with planning commissions and groups in regard to this particular idea. There are two cities which have extensive accomplishments in city planning. They are not alone in extensive accomplishment, but they are used merely for the purpose of illustration—Detroit, Mich., and St. Louis. Both have done an immense amount of work or at least have secured the execution of work by the city government. The Detroit Commission is named because it is probably clothed with more power than any planning commission in America. The charter of the City of Detroit gives the Planning Commission the widest authority. The Council cannot even initiate an ordinance relating to the opening, widening, or changing of the

location of a street, the acquisition of a park or of a piece of property for a public building; it must originate with the Planning Commission. The charter goes further and gives the Commission power to prepare and adopt a zoning ordinance, but it has not attempted to exercise this power, which it realizes is probably illegal.

What has happened? The Council has occasionally resented its deprivation of powers. By constant effort the Plan Commission has succeeded in keeping a reasonable degree of harmony, but without such effort the situation would have been extremely acute and has been at times with respect to certain issues. Yet Detroit, under it, has been able to accomplish a great deal—things which are not always spectacular on paper, but which have a profound total effect on the city at large.

As a contrast to that situation St. Louis has probably the loosest authority for city planning of any city in the United States. The present ordinance is the original ordinance creating the Planning Commission and was passed in the spring of 1912. All engineers know how limited the understanding and appreciation of city planning was at that time, yet the Commission has continued to function ever since with that particularly loose authority. Through intensive technical work, through incessant publicity, it has succeeded to the extent of securing a wide degree of acceptance and accomplishment.

The City of St. Louis has had ten years of experience, one illustration of which might be of interest to Mr. Bettman. Because of favorable public opinion for adherence to the city plan, the Planning Commission received a great degree of public support on projects that were not to the individual interest of certain political leaders. They began to throttle the work by breaking down the personnel of the Commission through purely political appointments. Fortunately, a change of administration came about in the nick of time. There are more ways than one in which the very best drawn plans may go awry.

The Planning Commission not having funds for extensive educational work, a citizens' committee was created to properly advise the public in the event the Planning Commission itself had been swayed into a political point of view.

One more illustration to show the power of public opinion: It is fundamentally important with respect to planning. Mr. Bettman realizes that despite all this authority of law or the provisions of the statute, if there is not an overwhelming public opinion constantly stirred up, constantly alert, sooner or later the plan will fail.

The illustration is this: Many are familiar with the St. Louis bond issue of $87 000 000 in 1923, among the items of which was a civic center. The bond issue included $5 000 000 for the acquisition of all the necessary land amounting to about 45 acres, about 7 acres already having been acquired; $5 000 000 for an auditorium; $4 000 000 for a court house; and $1 000 000 for a soldiers' memorial. These sums were nearly sufficient to complete the plan. There were only two additional buildings needed to complete the entire group.

A complete plan had been drawn. It had been published. Almost everybody voted, assuming that the plan was to be carried out more or less as pictured, but the plans of the Planning Commission were not in complete and final detail. An unnecessary amount of labor and expense would have been incurred without the approval by referendum and the voting of funds. The commercial element of the community opposed the location of the court house in the plaza, and organized to keep it in a particular section of the business district where it would suit their particular interests. There arose a referendum brought by the commercial interests to place the new court house in that section of the business district not in the civic center.

This rather long story is told because it is unique in American cities and is a refutation of the effectiveness of the Ohio method. With the Retail Merchants' Association, every down-town commercial organization, all the banks, and the political machine organized to keep that court house out of the civic center, a two-thirds vote of the Council would have meant loss of the most important building in the civic center. The citizens' committee composed of citizens from all walks of life, with almost no money to spend, with the support of two newspapers out of three, defeated the referendum and maintained the integrity of the plan by a vote of 2 to 1. Here was an example of the power and the importance of public opinion in the enforcement of the plan. The public would not have supported the plan except for the fact that it knew and understood. There is nothing more powerful than public opinion, and the importance of creating that opinion by the planning commission is in danger of being overlooked if too much reliance is placed on supposedly legal checks or safeguards, because sooner or later ways are always found to amend, change, or revise, unless public opinion blocks it.

Thomas H. Reed,* Esq. (by letter).—It would be ungracious, indeed, to seem to detract from the felicitations due to Cincinnati on her enviable acquisition of a legally established city plan. So much planning effort has got no further than glossy reports and elaborate diagrams, that a plan in force is a marvel to be heartily admired. Mr. Ford himself, however, has deliberately raised the general question of the wisdom of permitting planning commissions to make plans which have in effect the force of law. To this question the writer would return an emphatic negative.

One can readily pardon the city planning expert for exalting his specialty to a place of equality with the other aspects of city government taken together. Experts in education, public health, etc., are prone to do the same. The fact remains, however, that city planning is only one of many functions of the modern city. That a wise plan is essential to the successful performance of many of these other functions does not remove it from this position. It is no more essential to happy city life than many other services, such as those of the departments of health, fire, and police. There is nothing in the nature of the planning function itself to distinguish it from many other city

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activities. City planning is a technical art calling loudly for the leadership of experts but so are the construction of sewage disposal works, the development of transportation systems, or the conduct of diagnostic laboratories. Should, then, there be established for city planning a procedure at variance with that used for other municipal purposes?

An affirmative answer to this query obviously requires support by reasons of extraordinary cogency. These reasons must have relation not only to the problem of city planning, but to the infinitely more difficult and obviously more important problem of city government as a whole. It is entirely conceivable that a method which will give most quickly a well-established city plan, may not in the long run, prove consistent with the best management of the affairs of the city. Indeed, municipal reformers, during the last half century, have waged a furious battle against special jurisdictions—park boards, water boards, sewer boards, and what not—each of which was justified (at least, originally) by the necessities of a particular service, but all of which taken together have tended to destroy the responsibility of the general government of the city. If there is a conflict of interest between planning and sound principles of municipal organization any doubts should be resolved in favor of the latter.

In all systems of municipal government except American, the council has been the real governing body of the city and respected as such. In the United States, early in the Nineteenth Century city councils began a descent which resulted ultimately in their being deprived of most of their earlier powers and in the almost total loss of responsibility and public respect. No better example of this situation can be found than the position of the Cincinnati Council prior to January 1, 1926. It consisted of thirty-one members most of whom were entirely incapable of forming an independent judgment on the government and affairs of the city. In fact, the Council had almost totally abrogated the function of determining city policies, which might normally be supposed to belong to it, in favor of the Hamilton County Republican Committee. It was not until this tolerably enlightened but wholly extra-legal committee had placed the sign of its approval on it, that substantial progress could be made toward the establishment of a city plan. This Republican organization controlled absolutely not only the Council, but the Mayor and all his appointees.

It is not possible to do without a municipal legislative body. Somewhere the policies of the city must be determined, if not in a popularly elected and responsible body, then somewhere else. Very little support could be found for the proposition to abolish the city council. There is, however, but one alternative, that is, to put it on a respectable basis. It is not necessary to examine in detail the methods available for this purpose. It is enough to point out that the council cannot be made respectable by withholding from it legitimate legislative functions. In city government, administration of necessity occupies a more prominent place than legislation. The city is engaged primarily in rendering a multitude of services which involve few questions of general policy and innumerable questions of administrative appli-

cation. Under these circumstances it is no wonder that the city executive, be he mayor or manager, bulks larger in the public eye than the council. There is all the more reason for leaving with the council as fully as possible the determination of such matters of general policy as belong to city government.

City planning is obviously a legislative function. It involves the laying down of a permanent policy of city growth. Mr. Ford and other city planners will agree that it is the most important subject of municipal legislation. To remove it almost altogether from the competence of the council is to strike a deadly blow at the vitality of that body. It may well be admitted that city planning is a function for the performance of which a council needs technical advice. Advice, however, is essential to good legislation of almost every conceivable kind whether enacted by the Congress of the United States or by the council of the smallest city. The conditions of modern life have become so complicated that no body of men is capable of deciding well, on the basis of its own unaided discretion, any considerable number of the questions to which government gives rise. Until it is safe to leave each subject of legislation to a soviet of the experts in that subject, however, it is necessary to have legislatures. By no other means yet discovered can the interests of the community as a whole be represented in legislation.

The writer admits that the old Council of Cincinnati was entirely incompetent to deal with the plan of that great city. In another place* he has indicated his doubts of its ability to perform any representative function. This is by no means true of the Council of nine which took office in January, 1926. This Council was elected at large and is adequately compensated. As a result of the civic stimulus which often accompanies a new charter it is probably as able a council as ever directed the affairs of a large city in this country. In the choice and control of the city executive (a manager) it exercises powers which make service on it worth while to men of broad outlook. The way to good councils, in other words, is by increasing, not decreasing their importance.

Mr. Ford suggests an analogy between the adoption of a city plan and the regulations of a board of health, of a police board, or of a fire commission. The analogy is very weak. None of these bodies has anything but a very subordinte power of making regulations which must in every case conform to the State laws or city ordinances relating to the subject. These laws and ordinances are more often than not very detailed in character, leaving only a modicum of discretion to the board. A city plan is not only a more fundamental matter than the angle at which automobiles must park along the curb line; it is a subject with which State Legislatures have dealt only in the most general terms. If the council cannot legislate with regard to it, the discretion of a planning board, empowered as is that of Cincinnati, would be well-nigh absolute.

The Standard State Zoning Enabling Act, issued by the U. S. Department of Commerce, provides in its first section for vesting the power of zoning in the "legislative bodies of cities and incorporated villages", and it is rumored that

* "The Government of Cincinnati and Hamilton County," 1924, pp. 189, *et seq.*

the proposed Standard City Planning Enabling Act departs from this precedent and follows in substance the Ohio Planning Law. The Model Charter of the National Municipal League, however, representing the opinion of a large group of experts in city government, leaves the enactment of the city plan to the council. The opinion, thus given effect, is shared by a majority of political scientists. They would have little if any objection to requiring a two-thirds vote of the council to authorize any change in the plan not approved by the planning commission; but the original adoption of the master plan is a matter so fundamentally legislative that it can not without violating sound principles of governmental organization be vested elsewhere than in the council.

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Paper No. 1646

HEXAGONAL PLANNING, TRAFFIC INTERCEPTER, AND ORBIT*

By Noulan Cauchon,† Esq.

With Discussion by Messrs. John Nolen, E. P. Goodrich, Lawrence Veiller, George B. Ford, and Noulan Cauchon.

Synopsis

The problem discussed in this paper is congestion, the writer's aim being to present a study for the elimination of the problem.

Cause of Congestion

Status.—Congestion comes of ill-proportioned functions and inadequate services, nearly all variables from lack of definite purpose. Congestion comes of the lack of functional, organic "town planning," defined as "the scientific and orderly disposition of land and buildings in use and development, with a view to obviating congestion and securing economic and social efficiency, health and well-being in urban and rural communities." Congestion comes of inadequacy of living conditions due to shape and disposition of blocks, to over-density of population; of the scheme of streets and arteries being ill-proportioned in width, length, direction, and capacity.

Operation.—Congestion comes from the lack of comprehensive zoning for "use and development," for meeting occupational requirements of spacing, height, and bulk of buildings. Congestion comes from lack of zoning the density of population, that an organic plan of streets and arteries may be

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

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calculated in dimensions and correlated to the fluid—ebb and flow of population and traffic within the population-shed.

Desiderata.—An organic correlation of use, density, and dimensions will afford a normal interval of time-distance between home and work; will afford health, efficiency, and well-being. This implies fitting design to purpose in function and limitation; the planner's problem, if all remains variable, would be insoluble.

We must seek, therefore, to determine self-contained urban units of balanced functional ratios and the most efficient and desirable assembly and inter-relation of these units.

Analysis

Forces.—The forces of urban development are radial and circular and seemingly solution should be sought in approximation of the properties of circles—not in those of squares, as has been the traditional attempt to fit square plugs into round holes.

The principle of hexagonal planning, traffic intercepter, and orbit are submitted as affording a correlation of traffic that will obviate congestion and adjust environment to its human purpose—survival.

Forms

Blocks and Plan Pattern.—The circle is the most economic boundary (or street) to any given area, as a unit. Circular areas as units do not, however, assemble economically—they leave waste interstices. (See Fig. 3.) Allowing for flexibility and a measure of compression will, in practice, eliminate the wasteful interstices and develop the hexagonal form as the nearest approximation of the subdivision virtues of circles, yet free from their structural disabilities as streets. Below ground, sewers, mains, conduits, etc., must be built in straight sections; above ground, buildings, rectangular walls, etc., are built more economically and advantageously in straight sections than in curves.

A system of subdivision demands a pattern of units susceptible of continuous and indefinite contiguity without intersticial areas wasteful of local improvements and public services.

Three Main Conditions.—To obviate congestion of traffic, and the many evils that come of it, there are three main conditions to be solved:

1.—The distribution of population in its homes, for which hexagonal planning is submitted as being most advantageous. (See Fig. 1.)
2.—Transportation between home and work, for which the intercepter affords the optimum time-distance. (See Figs. 2 and 4.)
3.—Routing the process of "work", for which consider a correlated traffic orbit. (See Figs. 4 and 5.)

The Hexagon.—The hexagon unit block and pattern seems best to meet the requirements of a basic system and general pattern, viewing the major area of cities as residential hexagons, with ample latitude for specialized

differential and rectangular design to purpose of business, commerce, and industry. (See Figs. 2 and 4.)

The advantageous properties of the hexagonal residential block are:

A.—Health.—North pointed; universal access of sunshine; affords safety, rest, recreation, and amenities for child, adult, and community lift. (See Fig. 1.)

B.—Traffic.—Safety and acceleration; three-way wide angle of vision; street junction of three-collision points *versus* sixteen in rectangular limited-visibility crossings—collectively, total of eighteen *versus* sixty-four collision points. Further, the street length around the equivalent rectangular block is 1 420 ft., that is, an average collision point for every 22.19 ft., while the length around the hexagonal block is 1 284 ft., that is, an average collision point for only every 71.33 ft. of street length. Besides there is 10% less street length in the hexagonal street-planning system. (See Figs. 1 and 3.)

C.—Economy.—Fewer and shorter streets, less capital costs. (See Fig. 1.)

The psychological and actual properties of hexagonal planning may perhaps best be judged from the accompanying diagrams, particularly Fig. 3, and their indications, noting the correlated content and directness.

The Dominion Postal Authorities, at Ottawa, Ont., Canada, have kindly given consideration and thought to the effects which hexagonal planning would have on the delivery of mail, by letter carriers, on post-box collection by motor cycle, and on parcel delivery by truck. The opinion of the men in these different services is that they would find no difficulty in adopting their "bump of locality" to the hexagonal system of subdivision.

Previous Use of Hexagon Block

Mueller, in Germany, about 1909, advocated hexagon blocks, assembled so as to leave all sides as continuous straight streets; but this left triangular interstices which were economically wasteful of bounding streets and public services.*

A. R. Sennett, in England, in 1905, published a two-volume work on town planning, in which he advocated absolutely the rectangular circulatory system, even objecting strongly to the introduction of diagonals. He, however, advocated the subdivision of the rectangular blocks into lots on a hexagonal pattern, that is, three rows of lots between streets, the center row being small hexagons, off center to the street-fronting lots. The object of this disposition was to allow for equal frontage ownership, coupled with a choice of additional rear-yard area for those who wished to take such allotments.

Major Arteries.—There are examples in Europe; also in America. An hexagonal system of such arteries is shown in the 1807 Governor's Plan of Detroit,† which is the best example in America. The individual blocks in this

* "Town Planning," by Innigo Triggs, 1909, pp. 113–117.

† See Report, Plan Comm. of Detroit, Mich., 1924.

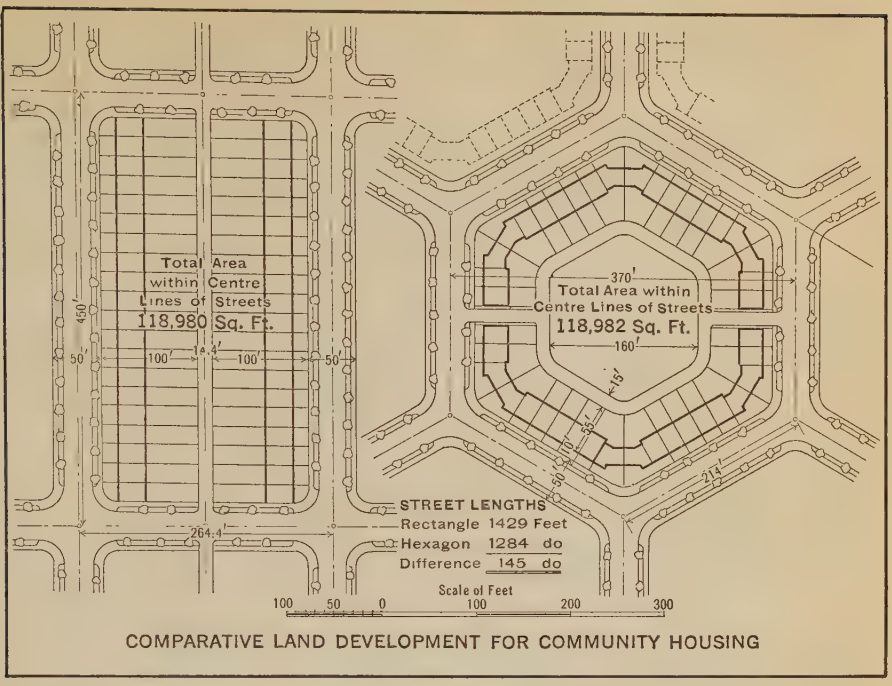


FIG. 1.—SECTION SHOWING PLAYGROUND OR GARDEN IN CENTER OF HEXAGONAL BLOCK.

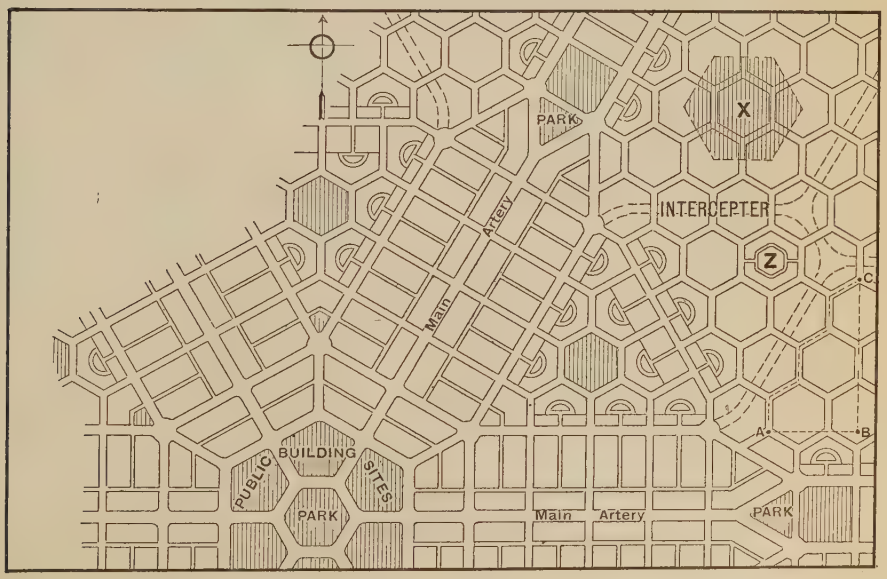


FIG. 2.—ILLUSTRATION OF RELATIVE BALANCE OF CELLS TO ARTERIES.

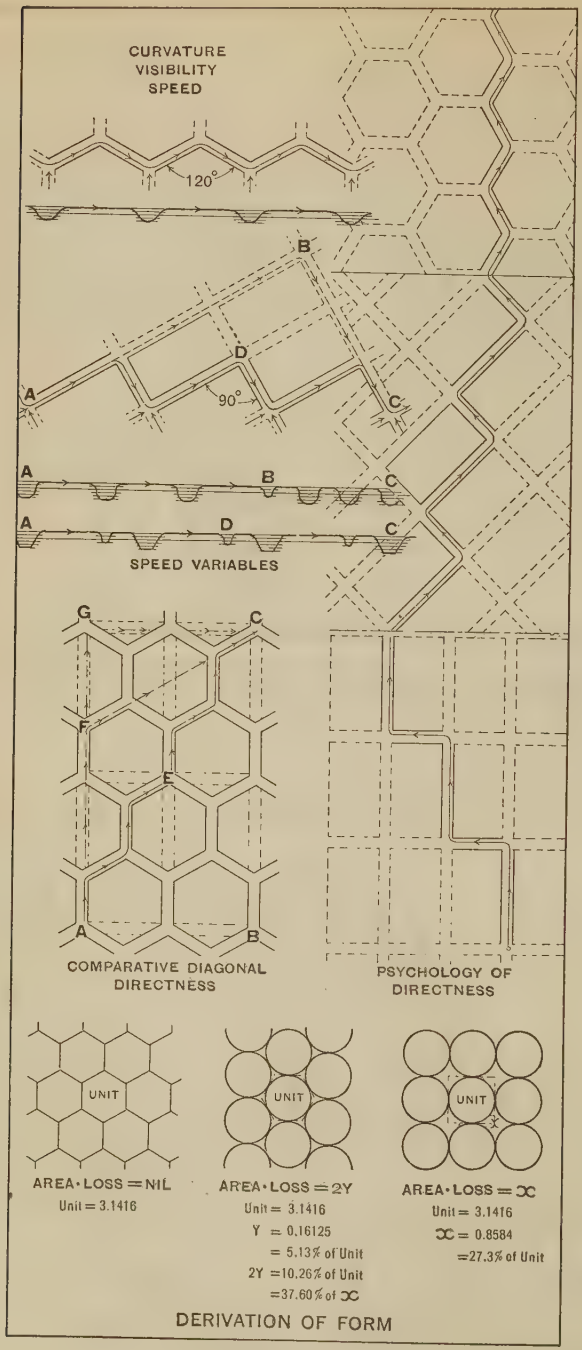

CURVATURE
VISIBILITY
SPEED
120°
B
D
A
90°
C
A
B
C
A
D
C
SPEED VARIABLES
G
C
F
E
A
B
COMPARATIVE DIAGONAL
DIRECTNESS
PSYCHOLOGY OF
DIRECTNESS
UNIT
AREA·LOSS = NIL
Unit = 3.1416
UNIT
AREA·LOSS = 2Y
Unit = 3.1416
Y = 0.16125
= 5.13% of Unit
2Y = 10.26% of Unit
= 37.60% of ∝
UNIT
AREA·LOSS = ∝
Unit = 3.1416
∝ = 0.8584
= 27.3% of Unit
DERIVATION OF FORM

FIG. 3.

Detroit plan are not hexagonal, they are rectangular and irregular and without any relation to advantageous orientation. Even with its limitations, however, it had many advantages over the then existing plans. Its abandonment was a great loss to progressive planning.

The adoption of a hexagonal block in basic patterns favors, although not necessarily nor exclusively, the development of a hexagonal system of major arteries, both surface and intercepter. (See Figs. 2 and 4.)

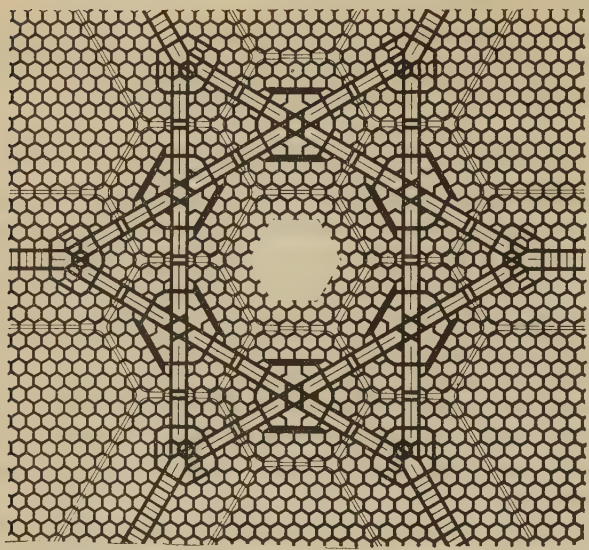
Collective Communications and the Intercepter

Minimum Time-Space Interval.—The requirements of communication demand of a plan system and pattern that it offer the minimum average interval in time-distance over a maximum area, between home and work and within the sum of objectives; that is, that the streets, arteries, and through transits should be so correlated that the tide of population and traffic may ebb and flow freely, without congestion, over the greatest possible area. This should be for a normal given time-distance of about 30 min., or so much more as psychology and circumstance may demark as the turning point of amenity in free attraction, competitive areas, and inducement to spread. (See Figs. 2 and 4.) The intercepter is shown in light parallel lines running through the small hexagons by grade separations.

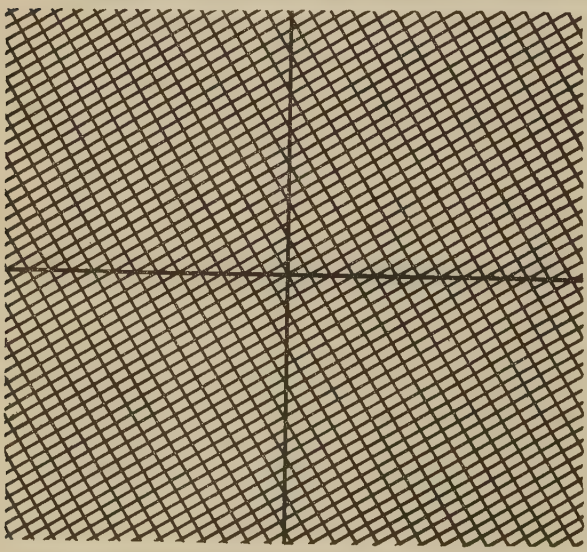
Present arterial provisions are inadequate for expansion because they ultimately become choked by cumulative local traffic. Arteries calculated for an initial purpose cannot be widened from generation to generation, as the process will eventuate in the logical absurdity of continually reducing the economic proportionate dimensions and area of the business lot on a thoroughfare of ever-increasing importance. Some new principle must be evolved for dealing more efficiently with the conditions of expansion which have come with modern civilization in greater speeds and vaster accumulations. That principle is the intercepter.

Intercepters.—The intercepting artery is one which carries through traffic free from cumulative local traffic; it is an artery on which no opening or traffic from facing property is allowed. While denying accumulative access along its frontage, it nevertheless contributes open spaces for sunshine and air to the windows of that frontage. The adits and exits of such an intercepting artery should be placed at relatively long distances apart, as are stations on a railway. It is recommended that the distances between the adits and exits of an intercepting artery be about ½ mile apart.

A correlated system of environment can only be designed scientifically for given purpose and capacity. Accretion (*laisser faire* policy) beyond the designed given organic entity of correlated parts and functions (ratios of forms and communications) will cause congestion and its disintegrating processes. Organic expansion beyond the initially designed capacity of the civic machine should be taken care of on the principle of satellite sub-centers. Interrelations between the central area of attraction and its satellite sub-centers should be provided for by intercepters, that is, rapid highways free from the accretions of local frontage access.



HEXAGONS, INTERCEPTER, AND ORBIT.



RECTANGULAR GRIDIRON AND DIAGONALS.

FIG. 4.—DIAGRAMS SHOWING COMPARATIVE CIRCULATION.

The factors of population density, as determined by zoning (use and bulk), and as provided for in capacity of width and cumulative length of streets and arteries, with an index of speed at which the population-shed can mobilize its fluid content, determine the time-distance radii of a normal civic entity. This should be the limit of the self-contained civic entity if it is to attain and retain the optimum in human, social, and economic efficiency of the individual, in city and State.

Average straightness between the total of objectives may be summed up as general directness. Directness is also conditioned by visibility accelerating speed. Further, there are factors of psychology in directness, the appearance *versus* the reality. (See Figs. 2, 3, and 4.)

Cost

Relief versus Congestion.—The question of the cost of land required for intercepters and the cost of their construction is in principle answered by comparison, on its merits, in given circumstances, with the cost of congestion, where lack of width and freedom develop it. And what does it cost to relieve it by widenings, subways, viaducts, etc., prohibitive as a general process?

Where the relief by widening, etc., is only local, it frequently merely defeats its own ends by inducing a diversion of traffic to itself and renewing congestion, but to a more intense degree.

Individual surface diagonals by their more outstanding directness, where not part of an organic system, are also apt to develop congestion and contingent disabilities, although they are often the lesser of two evils.

The Gridiron

The gridiron street system is a relic of primitive two-dimensional thinking, merely length and breadth. It develops the illusion that from any street intersection in any area there is a straight way to all other objective points within the area. Whereas the gridiron favors straight ways in merely two directions, it presents the long way round, two sides of a triangle instead of the diagonal, to the great majority of destinations. (See Figs. 3 and 4.)

The illusion of general directness in the gridiron has been fostered by the coincidence that city maps are conventionally made heading to the north, by which the apparent mass of streets is leading freely out and away from the observer (look at Fig. 4 according to north point); the eye seems more inclined to sweep the vertical plane and with less effort than the horizontal. When a gridiron plan is studied at an angle of 45°, cornerwise, for traffic relief, the indirection of routing arouses an almost instinctive longing for diagonal directness. (See Fig. 4.)

The thoughtless license given to population density by heights abnormal to the channels of communication awakened engineers to a third dimension, namely, volume. Very soon recognition of time will follow—time-distance in speed and safety—as the qualifying fourth, the limiting one of capacity.

A balanced correlation of forms and contents, of channels and speeds, must now be sought, the norm of entities and inter-relations that will afford freedom from congestion; for congestion is the cancer of community progress.

The Orbit

In an accompanying schematic diagram (Fig. 4) a field of hexagonal residential blocks and streets is shown, served by a twin system of surface arteries for local distribution and diffusion, circumscribing a large open central area. The mobilization of traffic throughout the population-shed is compensated and accelerated at "vortex" contacts between the local arteries and the through-traffic intercepters. (Intercepters shown in light double line, Figs. 2 and 4.)

The diagram of comparative circulation (Figs. 4 and 5) is to illustrate a theory that major arteries in converging, as they should, toward a central area, still ought to diffuse far short of a central point. There should be also a relatively large park-like inner central and pivotal area of low pressure; further, that there would be engendered by a design of this nature a constant disposition to division and deflection of incoming and outgoing movements, establishing balanced centripetal and centrifugal mass movements of traffic, that would set up a form of gravitational attraction swaying and converting the inevitable peaking of the focal surge of traffic into rotation on a well-balanced orbit.* (See Figs. 4 and 5.)

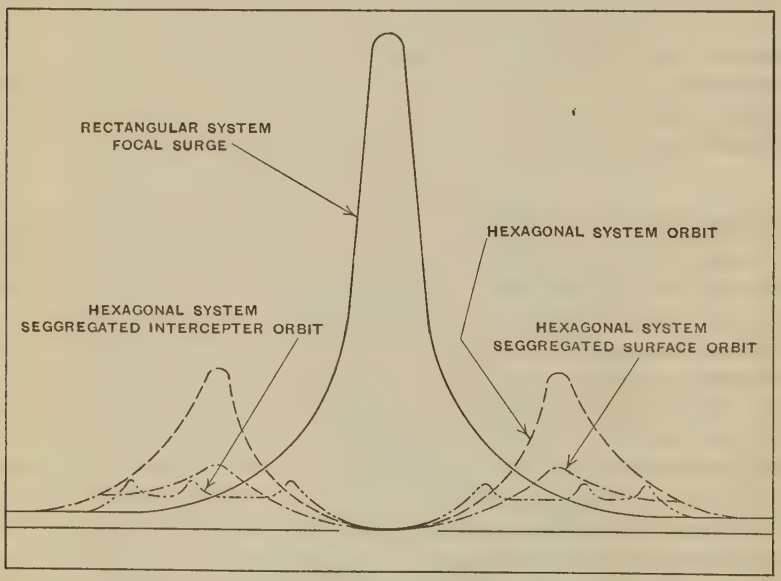


Fig. 5.—Orviation of Traffic Congestion. Comparative Cross-Sections.

Instinct of Survival.—The instinct of racial survival has engendered a dominant sense of community welfare justifying the view of the Hon. W. L. McKenzie that "private rights should cease when they become public wrongs"—the formula of liberty under which we should operate—all government being but synthetic organization for the maintenance of life.

* Compare comparative cross-sections of focal surge peak *vs.* gravitational orbit.

Research is required to determine the "norm" of adjustment for survival within the municipal machines in which we live—if the bearings be expected to run smoothly and the fly-wheel of progression to be kept from disaster. The civic problem, on the *laisser faire* method of indefinite variables, is evidently unsolvable.

The basis of planning must be a healthy maximum relative density of occupational living conditions; and, as a principle of racial survival, the point of permissible saturation must not exceed that of social well-being. In the ultimate higher economics of Ruskin, "there is no wealth but life"—anything less is only a measure of commercial cannibalism.

DISCUSSION

JOHN NOLEN,* M. AM. SOC. C. E.—The profession is indebted to Mr. Cauchon for the constructive thinking that his work represents, and the combination of fresh ideas related to old ideas, and new terminology for well-recognized principles in science and art, design and construction.

The one thing that is particularly significant is the presentation of new plans and technique to meet the new conditions, standards, and requirements, of the world to-day, and the necessity of recognizing that broad part of the field of planning which is not merely the patchwork and piecemeal system of doing what we can, what we must do, with existing cities.

This paper involves somewhat the by-play of planning cities according to preconceived plans. Triggs, in a book written about 1909, divided cities, under preconceived plans, into three classes which he called the spider's web, or the radial system, the rectangular or chessboard, and a combination of the two classes. In other words, he practically limited it to those classifications, and stated:

"In nearly every case these are founded upon some geometrical figure, a square or triangle, or the favored hexagon. * * * But such schemes, though they appear well on paper, are almost impossible to carry out in reality, because they could only be carried out on a level plane, and they pay little or no regard to the natural features that are generally the prime reason of a city's location. At the same time such ideal plans have their value. The most successful form that has been yet devised is the hexagon, of which type we illustrate two examples, both having certain points to commend them. It is claimed for the hexagonal type that it permits the development of the city to the utmost that might be possible within many decades, because, with the hexagon, the great advantage of the diagonal line is secured and at the same time many intervening spaces are provided that are so much in demand in all city plans."

An interesting comment in this general field is what might be called the "bee-hive" type, in "The Spirit of the Hive", by Dallas Lore Sharp. There is also the old "roadtown" type by Chambless, which is another scheme. The "Regional Planning Theory" by Arthur C. Comey, M. Am. Soc. C. E., is a reply to British town planning. The satellite towns program is particularly well illustrated in plans drawn by Messrs. Raymond Unwin, Robert Whitten, and others.

Arterial highways and hexagonal planning are the combination and recognition of the traffic problem in connection with hexagonal planning. Mr. Cauchon advocates a principle of through intercepting arteries, free from local traffic, as opposed to the present practice of enlarging existing arteries; also, that new residential subdivisions should be on the hexagonal pattern and obtain their through-traffic service by intercepting arteries.

There is the fundamental question of whether a city can be planned as a project with any sense of completeness, in its whole area and with regard to its whole purpose. It must be recognized that the biological principle of growth is in the nature of a city and that it must carry with it opportunities

* City Planner, Cambridge, Mass.

for reasonable expansion and change of function, and that is why such planning comes to be rather on the side of art than of science. In attempting to make city planning mechanical and to standardize it, except as a principle of organization, limitations and trouble are immediately invited because that is not the nature of either the city or of human life. Paradoxically as it may seem, the most permanent thing is change. A scheme which is not elastic enough to have this dynamic element of change, will soon show its limitations. This does not mean, however, that every design of a railroad, or building, or bridge, or city does not, in some way, try to bring within the human limits of possibility the forecasting of conditions over which they may have control.

E. P. Goodrich,* M. Am. Soc. C. E.—Mr. Cauchon has pointed out with a great deal of accuracy most of the fundamental principles which must underlie a scientific design of any community. With all due regard to those who are the predecessors in the working out of the art of the planning, it is the science of city planning that is going to do more in the future to create the better living conditions, the cheaper traffic operations, and the easier running of this big machine than has been possible under the development of the art so far as it has gone to date. This is one of a series of suggestions in the development of that technique. It is a very interesting start, and the mind naturally reverts to how the present gridiron type, which is almost universal, can possibly be adapted to produce the same general effect that Mr. Cauchon has suggested. For example, his introduction of an interior play area into the center of a block is obviously possible in a rectangular system exactly in the same way in which he has developed it in the hexagonal scheme. Even his playground is simply the working out of such a scheme as was actually developed in Washington, D. C., for the improvement of one particular block in its interior. Thus, there is no real virtue in the hexagonal shape as far as that is concerned.

Substantially this system, as Mr. Cauchon indicated, was developed in 1803 in Detroit, Mich., around the central part of the city, but as he has suggested it has been abandoned, and traffic requirements have dictated for economic reasons the cutting through of various other direct routes so as not to break up the traffic as has occurred heretofore, but to get it away as fast as possible. It has been found that this system in Detroit actually congests traffic more than the direct route system which has been created in other parts of the community and is now being superimposed on that rather extra length system in the down-town section. Furthermore, the people who live in this particular area are the only ones who know how to get out of it. A foreigner, a non-resident, always gets lost in that part of Detroit, and even if he is from the outlying sections he cannot go through it with any degree of certainty that he is going to "arrive". While theoretically there is an advantage, practically there is a great disadvantage in that type of street layout.

Mr. Cauchon believes that the radial and the circumferential idea is the one that should apply in the design of a city structure. If to that general

* Cons. Engr., New York, N. Y.

statement were added the condition that it applies to the natural nuclei of the community, it would be generally accepted. An examination of traffic actually moving in different communities, for example, in Cincinnati, Ohio, or Norfolk, Va., where special studies were made, shows that if areas are selected with some reference to traffic origin, such units create traffic in certain proportions depending on the residential character, the density of the population, and the use of the streets for business and industry, and that there is a desire on the part of the people in one particular unit to go to each of the other units in proportion to the inverse ratio of the distance between those different sections. There should then be in any proper design, a system of what might be called spider webs connecting the nuclei as directly as possible. That will take the traffic from one point to another rapidly. It will reduce the cost as far as traffic is concerned. It will reduce the frontage which must be devoted, in part, to the use of through traffic. Between these various spider webs, possibly the hexagonal system might be applied to advantage. It is also possible to make varieties of interior arrangements as irregular as one chooses so as to provide the same friction to through movement in those sections. Many designs have been submitted and actually laid down on the ground in various sections, St. Louis, Mo., for example, where the specific object has been to prevent the use of such streets by through traffic. There is small tendency to move through such developments, however, if the nuclei are connected as directly as possible. It is entirely feasible in the connection of these various sectors or elements to make a connection by one device or another from the intermediate interior streets to the main thoroughfares.

Mr. Cauchon has very strongly pointed out the need of sunlight in modern city development, and has suggested that the hexagon will provide the best possible arrangement for the street system so as to secure the utmost of sunlight in rooms of the buildings on his frontage. All agree that sunlight is important; in fact, that it is the most important element possible, not only in residential planning, but in industrial and even in commercial planning; that it will be in the future more and more appreciated; and that the future scientific design of cities will probably take into account sunlight planning to a greater extent than ever before.

The speaker must, however, dissent from the contention that the hexagon is the best shape. Fig. 6 is a sun dial designed for the latitude of New York. It gives the length and direction of a shadow at any day and hour. For any community of a different latitude the curves would be somewhat modified. The north point is vertical. At noon, the shadow would go from the center of the circle to any one of the different curved lines, depending on the time of the year. At the winter solstice the shadow would be from the center of the circle to the upper of the curved lines. At any time of the day the shadow would go from the center to the point on the proper one of the curves which is intersected by the straight line giving the hour. With this sun dial one can lay down a street system and see whether he can get sun into the rooms facing on certain streets.

Using this sun dial, studies have been made of the period over which the sun would penetrate a window facing in each direction on the shortest day of the year. After studying the matter, it was felt that a 30-min. interval of penetration was the minimum which should be adopted. A 1-min. penetration is insufficient to secure any curative effect of sunlight of any consequence, or any of the other benefits which direct sunlight penetrating a room has been shown to have from a medical standpoint.

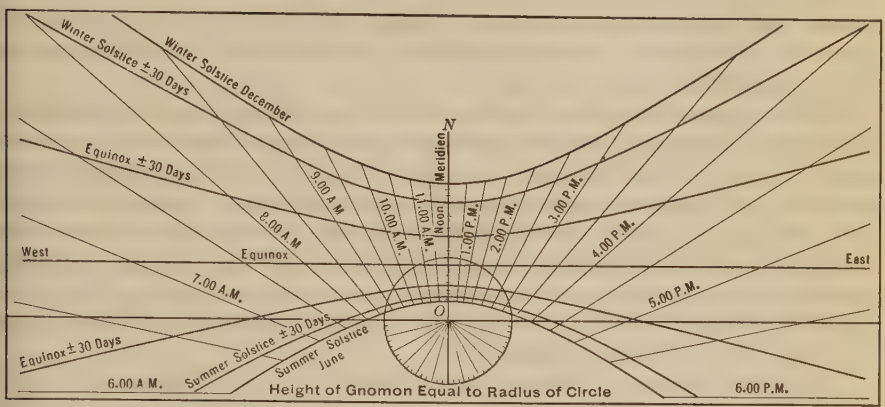


FIG. 6.—LENGTH AND AZIMUTH OF SHADOW OF UNIT GNOMON CAST BY SUN AT ANY HOUR OF DAY AND AT ANY TIME OF YEAR, LATITUDE 40° 45′ NORTH.

The advantage in getting the sunlight into the street is small compared with the need of getting it into the room. The simple fact of the sun shining along the street from time to time, is less than 10% as valuable as letting it into the dwelling. It can be shown by the use of the sun dial and the penetration figures, that a properly oriented rectangular street system is much superior to the hexagonal scheme from a sunlight standpoint.

Mr. Cauchon has suggested the need of intercepters, which are essential; but if the intercepters are to maintain the hexagonal routing there applies to them, as also to the street system of the small hexagon, the question as to distance and the economics of traffic.

In Fig. 4, if a person wanted to move from a point at the lower end of one hexagon directly east, obviously there is an extra distance in the ratio of 100 to 75 compared with a straight line. The straight line is 75% of the distance along the hexagonal. Likewise, if one wanted to move south there would be the ratio of 100 to 87. If one wanted to move in a direction across the hexagon parallel with one of its sloping sides, and in place thereof had to go around a rectangular block, the ratio is as 102 by the rectangle to 100 by the hexagon. If a person wanted to go south 30° east, the hexagon compared with the rectangle is as 100 to 138. In a 45° direction the ratio is 103 rectangular to 100 hexagonal. In other words, in going around the circle, if one happens to strike the direct route of the rectangular system of straight streets, about 25% has been gained. If one wants to go diagonally, the hexagonal system is shorter by between 2 and 3% up to 38% in one instance. So that, gener-

ally speaking, it is disadvantageous to move by the hexagonal route compared with the rectangular one by a margin of 10 or 15%, which, because of the fact that the cost of operation of the system is so much greater than the interest on the investment, will overcome the benefit of the shorter length of streets with reference to area which obviously applies to the hexagonal system.

There is the further difficulty, that some day, whether it be on the rectangular or on the hexagonal system, traffic regulation will be arranged so as to be continuous at all times on every street. It will not be a stop-and-start system, but it will have continuous motion. That can be worked out in two ways with reference to the rectangle, but when it comes to the hexagon, it seems impossible.

With the rectangular type, traffic can be in one direction on one thoroughfare and in the other direction on another, moving in blocks so co-ordinated or regulated that traffic can move continuously on all the thoroughfares; or, similarly, by designing streets in pairs or in certain other schemes, all traffic will move on north and south thoroughfares continuously at all times, and on the east and west thoroughfares block by block. If that scheme is worked out and the north and south thoroughfares are designed to be three times the capacity of the east and west streets, it will be found possible to move in an east and west direction continuously.

The whole problem of street layout design is one of economics of administration, based on such a scheme as will produce the best possible acquisition of sunlight and traffic feasibility in any community. As between the rectangular and the hexagonal systems, it seems at the moment that those two elements are in favor of the rectangular rather than the hexagon, in spite of the brilliant suggestion which Mr. Cauchon has made.

Lawrence Veiller,* Esq.—Mr. Goodrich has subconsciously applied the author's plan to an existing, built-up community, and especially a great city like New York or Chicago, and Mr. Cauchon, at the very outset and in all presentations of his plan heretofore, has absolutely disclaimed any such idea. He has said this was a scheme for the development of a residential community on the new parts of an existing community. He has also tied to it the scheme of intercepter highways.

Mr. Goodrich's presentation does not detract in any way from the scheme, but on the contrary seems to re-endorse the arguments as presented. All the questions that he raises as to these different foci of interests and attractions apply to an existing community, because when a new town is built they do not exist. There will be other foci, but Mr. Cauchon's scheme takes care of them. So that argument falls to the ground as far as it relates to the new community, but it does apply to existing communities because there are tendencies already developed that cannot be changed.

Again, with regard to the question of regulating traffic which Mr. Goodrich rightly points out, using the two diagrams of collision or confusion points: There will be no necessity to regulate traffic if these residential communities are developed. There will be no police signal systems, for there will be no such

* Secy. and Director, National Housing Assoc., New York, N. Y.

plans. The whole thing is scientifically planned to discourage that. Mr. Goodrich was no doubt thinking in terms of cities like New York which have the gridiron system where traffic has to be regulated.

GEORGE B. FORD,* ESQ.—Two questions arise about the extremely interesting subject which Mr. Cauchon has presented.

In the first place, with regard to the shape and size of individual lots: Would not a more varied scheme—a combination of winding roads and straight roads—allow fully as much space per individual householder and at the same time give more interesting shapes to the lots and more variety than the hexagonal scheme could possibly give?

In the second place, in looking at the general hexagonal plan one gets the impression that the total amount of space used in streets in the hexagonal scheme plus the superimposed thoroughfares is actually greater than the total amount of space that would be used in a rectangular scheme even with diagonal arteries superimposed on it, particularly on account of the number of half-blocks, and quarter-blocks that are left where the hexagonal scheme joins into the thoroughfare spider web, and on to the rectangular street layout.

NOULAN CAUCHON,† ESQ.—In regard to the size and shape of the lots: The basis of all planning is the technique of sociology. The standard lot which is used, is 100 ft. in depth, and varies in width. A great deal of it is needlessly put into backyards only partly used. The number of attempts to make collective playgrounds of these backyards are evidence of the fact that they are little needed in their present form. In the hexagonal block the backyards, or those parts of the backyards, that can be spared for recreational purposes, are assembled in a more advantageous way than can be done in the longer rectangular block.

As to uniformity, the hexagon is not limited to any exact size, neither is it essential that it should equal some particular rectangular block. If a reasonable area for a unit could be agreed upon, say, 1 000 sq. ft. per individual, this, transposed into a lot 125 ft. deep, would produce more backyard. The difference between these two in respect to the shape of the lot is the purely social advantage in having the surplus backyards aggregated in the hexagonal form. In zoning, about 30% of the lot is allowed to be covered by buildings. It is immaterial to the lessor whether the remainder of that lot, 70%, is a backyard or a fine playground.

As to the artistic side, it is not necessary that this should be carried out everywhere. There is no reason why certain areas should not be treated in a landscape manner, with winding roads, especially down hillsides where there are good engineering reasons for having a winding road. It is simply suggested as a basic pattern for residential districts, not for business districts. In the business sections, right angles are retained in order to facilitate grade separation and traffic control.

In regard to the other point Mr. Ford raised, the imposition of these large traffic arteries certainly takes up some of that 10%, but to advantage. The

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† Pres., Town Planning Inst. of Canada, Ottawa, Ont., Canada.

10% advantage is an advantage purely in the length of the street. A field of hexagons would discourage through cumulative traffic, and residential streets could then be reduced to possibly 30 or 40 ft. in width. Farther away from the intercepters they do not have any great load of traffic to carry; there is practically a country road seclusion. What is taken off the width of residential streets can be added to the main traffic areas, and still be several per cent. ahead of the present scheme, with a better traffic arrangement.

In regard to the question of housing: New York has a State Commission on housing, and its report (Document No. 11 925), reveals officially that practically two-thirds of the working population, below a certain level, cannot be provided with housing for themselves on an economic basis. In other words, the cost of living is so great that the laborer cannot provide himself with decent living conditions, nor will the costs of money allow of its provision in the State of New York. There is something radically wrong. That report is considered one of the finest public documents ever published, as it reveals the condition to be faced. It is an economic condition which, if it cannot be helped, simply means degradation to the class of people which cannot earn enough to house itself or be housed decently. Such conditions have to be changed or the race will die out. One of the advantages of the immigration laws is that they are going to raise the standard of those coming into the country, but, if a condition is to be maintained whereby the population in the country cannot be self-sustaining on the basis of decent living, it means the annihilation of the race.

There is a cost, and a justifiable one, of survival; a person or a group of persons cannot be allowed to have the power of throwing the cost over on the remainder of the community; thus pauperizing them to live at the cost of the other people. Referring to the question of slums, the following illustration is applicable: the cannibal cooks and eats an individual while he is fat; the commercial cannibal imposes living conditions which gradually disintegrate the human frame; he makes his victims skin and bones before he eats them; he practically lives on their blood. It is a matter of degree, but that is exactly what is happening, and the State of New York or any other State, or any Province in Canada, which has a growth that is going to destroy the life of the community must have such malign growth extracted—to begin by the abatement of congestion.

The diagram (Fig. 4) illustrates the point that there should be no centralized concentration. The rectangular system on a limited scale is not harmful. One of the psychological troubles with love for the unlimited gridiron is that city planners are accustomed to it and think every man should have a thoroughfare straight where he wants to go. An attempt was made here to design something which would dissuade the public from the use of residential streets as through thoroughfares. There is no reason why every one should require to go straight through to somebody who lives five or six miles away. It would be easier to drop in on a general through street which is wide enough for all traffic. Evidently one cannot design something that will adapt itself in all shapes and forms for everything that is wanted. That is the trouble with the

present plan, the assumption of illimitable convertibility as desirable. In Fig. 6, Mr. Goodrich is evidently thinking in terms of through traffic.

The intercepters happen to develop that way. There are six ways out of the hexagon and it gives that much greater flexibility and discretion. It fits. The intercepter can be put in any of these directions. Keep out of mind that it is a fixed diagonal; through traffic like surface traffic should adopt a path of least resistance, that is simply to be applied by the engineer who has experience to fit it to topography. It is not a case of creating a plan, but of finding what is the best way out. The plan is not being imposed upon the topography. If through traffic can be dissuaded from going indiscriminately through the hexagonal field and keep it in the intercepters, there is no need for great width of streets for there is no cumulative traffic. An inquiry: What is the proper width for a street? It somewhat depends on how long it is to be. If it is 10 miles long the street might be sufficient if it were 50 ft. wide; if it were shorter it might be necessary to have it 60 or 150 ft. wide; according to the density of the adjacent and cumulative population and traffic sheds.

Then, there is the question of sunlight. It is not claimed that divergent streets are as good of themselves as north and south streets, but it is claimed that the top and bottom of the hexagon is a great deal better than an east and west street, and you cannot have them all north and south. The generalized condition of the divergent and short length streets, coupled with the openings of the hexagon, allows of a greater sun penetration on the average throughout the entire system. There is a big building in Ottawa, a type of which bankers and insurance companies are fond—with deep walls and big pillars—which faces north. If the winter sun ever shines on the north side it will never be into the windows. That is not a rational way of planning. A style of architecture which is beautiful in the south because it is a protection against the sun is not beautiful in the north where it lacks the sun. The ideal is only ideal in the measure of its fitness for purpose, and a thing that is not ideal is not beautiful.

In Ottawa the only decent streets were built one hundred years ago by Colonel Bey, the Engineer Founder of the city. A statue to Colonel Bey is to be erected by the Dominion and it will be the first statue erected in Canada to an engineer. The town planning he did was this: He simply visioned wide streets; he made a number of streets 132 ft. wide. They are the only decent streets in the city.

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Paper No. 1647

GARBAGE DISPOSAL
A SYMPOSIUM*

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* Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 21, 1926.

ADMINISTRATIVE AND ENGINEERING WORK IN THE COLLECTION AND DISPOSAL OF GARBAGE: A REVIEW OF THE PROBLEM

Samuel A. Greeley,* M. Am. Soc. C. E.

Synopsis

This paper describes briefly some of the administrative and engineering problems in projects for the collection and disposal of garbage. Technical literature occasionally states that the disposal of garbage has not kept pace with other sanitary engineering works. Such statements are generally coupled with the suggestion that closer adherence to competent technical guidance would greatly improve the results. This is a sound suggestion. Some of the troubles are inherent in the situation and will yield finally only to general public opinion.

The paper outlines relative costs for garbage collection and disposal and other sanitary engineering works. Typical procedures for the acquisition of garbage disposal plants are discussed, contracts and specifications outlined, and engineering items in garbage disposal listed.

General Statement

This paper relates briefly to some of the present-day administrative and engineering problems in the development of projects for the collection and disposal of garbage. Occasionally statements appear in technical journals and engineering society proceedings that the disposal of garbage has not kept pace with other enterprises dealing with the public health and comfort, commonly termed sanitary engineering works. Such comparisons inevitably involve remote influences as well as intimate reactions. Criticisms are generally coupled with the suggestion that closer adherence to competent technical guidance would greatly improve the results; which is, of course, a sound statement. Some of the troubles are inherent in the situation and usually will yield only to general public opinion.

Works for the collection and disposal of garbage involve much lower per capita construction costs and require a much larger personnel in operation than other sanitary engineering works, such as water supply and sewerage. Thus, an opportunity is relatively easily acquired to employ a comparatively large number of unskilled workers. Collection equipment, and even garbage disposal plants, can usually be financed without great infringement on the public credit and, therefore, often without the necessity of general public attention and support. Inherently, therefore, the interest in the collection and disposal of garbage is measured by different yard-sticks than those applicable to other sanitary engineering works.

* (Pearse, Greeley & Hansen), Chicago, Ill.

Thus, in one community where the writer has been engaged for the last decade, garbage disposal works are estimated to cost about $75 000, whereas $2 500 000 has been spent on an additional water supply and $2 000 000 is being spent on sewage disposal.

A number of other less fundamental troubles may be briefly stated:

First.—Garbage disposal works proper have been sometimes acquired by municipalities under general service contracts or by purchase under general specifications which require the fulfillment of certain guaranties. Thus, the design, as well as the cost, becomes competitive, and improvements in the design resulting from experience are confined by the exigencies of a somewhat promiscuous competitive field. Manufacturers of garbage disposal equipment of the better class are restricted in their developmental work by the hazards of the competition of low-priced and low quality equipment.

Second.—The selection of a method of garbage disposal from several satisfactory processes is occasionally left to a consideration of competitive bids received under specifications which define the work to be done, but which leave both the choice of the process and the details of design to each bidder. Under such circumstances the selection of the "best" bid is not easy and the choice of a process, as well as the quality of design included in the bid, are likely to be oriented somewhat to the limits imposed by the competitive conditions. Such a procedure should not be confused with the general service contract under which the city does not acquire the plant, but contracts for the service of garbage collection and disposal.

Third.—The award of a contract for a garbage disposal plant generally leaves the final acceptance of the work to the fulfillment of certain guaranties to be determined by tests. Frequently, the making of satisfactory tests is hard because of the difficulty of accumulating a sufficient supply of the desired quality of garbage and rubbish. The value of the tests as an index of normal operating efficiency may be questioned because test runs are sometimes made with specially skilled labor. The rejection of a plant on the part of a municipality is likely to mean not only litigation, but also a delay in the solution of the garbage disposal problem. Hence, even if the rejection is upheld in the Courts, there is a large and undesirable economic loss. Uncertainties such as these tend to introduce a gambling element which may attract an undesirable type of contractor, and thereby work against safe and conservative design and construction.

The purpose of this paper is to throw some light on these items. Broadly speaking these inherent troubles are limited more or less directly in accordance with the amount of skilled and experienced engineering effort applied to the problem. Such engineering work may be, and often is, supplied by general contractors, by city engineers and other municipal officers, and by consulting or practicing engineers. Generally, the quality and freedom and not the source of the engineering work are important. Improvements in the garbage field are most likely to come from a better understanding, on the part of the general public, of the problem and its inherent difficulties.

Cost Aspects of Collection Work

The collection of garbage is largely a problem of administration and organization. The engineer is concerned chiefly with the collection equipment, such as wagons, motor trucks, trailers, stables, and garages, with considerable opportunity for productive work in its selection and the subsequent routing and operation. The actual design of wagons and trucks, however, is done largely by the manufacturers, so that finished products are commonly bought under general specifications. Very often such specifications and methods of purchase do not permit a close and actual money comparison of bids.

In a city of several hundred thousand people, the equipment for garbage collection may represent an initial investment of perhaps 30 to 40 cents per capita. As compared with this, works for the collection of domestic sewage may cost for construction, one hundred times as much, or $30 to $40 per capita, depending on the concentration of population, the topography, and similar features. The operation of the garbage collection equipment, however, is much more costly than the maintenance and operation of a sewerage system. A city of 400 000 people may need 50 to 60 garbage collection units (exclusive of ashes and rubbish), with 125 to 150 workmen. Sewer maintenance requires probably less than 25 workmen. Thus, a relatively low cost equipment can be secured, which provides work for a large number of men.

Cost Aspects of Disposal

The disposal of garbage involves administrative and engineering problems. The design and construction of a garbage disposal plant is essentially a problem involving both civil and mechanical engineering. In a city of several hundred thousand, the construction cost of a complete garbage disposal plant may be from $0.75 to $1.00 per capita. For comparison, the cost of a complete sewage treatment plant is likely to be ten times as much. The operation of the garbage plant, however, will require three to four times as many men as might be required in the operation of the sewage treatment plant. In projects for the disposal of garbage, therefore, as compared with the disposal of sewage, the first cost is relatively low, and the operating cost and personnel are relatively high. Much the same comparison holds between garbage works and other municipal enterprises, such as water supply, paving, etc. The collection and disposal of garbage is one of the least costly municipal enterprises to acquire and one of the most expensive to operate. This emphasizes the administrative aspects of the work, as compared with the engineering.

Relative Costs of Collection and Disposal

Not only is the collection and disposal of garbage more expensive in annual operation than most other municipal enterprises, but the collection part of the work usually involves by far the larger cost and greater personnel. Of the garbage budget 75% may easily go to the collection work. As the collection of garbage is so largely administrative, its greater importance on a cost

basis tends to detract from the consideration due the engineering aspects of the work as a whole.

Present Practice as Regards Incineration

At present, incinerators for garbage disposal are generally acquired on a "duty" or performance basis. After funds are available and a site has been selected, and perhaps acquired, the city advertises for bids on a general specification requiring each bidder to submit his own plans. Occasionally, the specifications are brief, stating only the capacity desired and perhaps the height of the chimney. More extensive specifications cover the materials and include guaranties of operation. More recently, plans have been prepared detailing the general layout, the building construction, the chimney, the approaches, and similar items. This limits the bidders' plans to the incinerator proper, reduces the items not easily compared on a money basis, and thus tends to a more equitable letting. Sometimes the type of incinerator is specified, that is, whether it shall be of the inside or outside storage type, and the limiting areas of the grates, flues, combustion chambers, and other parts are stated. In some specifications, only incinerators with a service record of a stated period of years are permitted. This tends to limit the field.

The specifications used in Milwaukee, Wis., in 1909 required the bidders to guarantee an efficiency and an operating cost, and the bids were compared on a computed annual cost. Thus the service character of the project was frankly embraced. More recent specifications have not included this feature, perhaps because of the difficulty of measuring practical operating costs under test conditions.

A report outlining such general specifications has been prepared by the Committee on Refuse Collection and Disposal of the Sanitary Engineering Section of the American Public Health Association.* Such reports are helpful in promoting the detailing of the incinerator specifications to the utmost under present competitive conditions and patent limitations.

Outline of the Incinerator Specifications

Under some specifications for a garbage incinerator prepared by the writer, bids were asked for under five items as follows:

(*a*) Incinerator furnaces with all appurtenances;
(*b*) Incinerator building and scale;
(*c*) Chimney;
(*d*) Runway; and
(*e*) Sewers and sewage disposal plant.

This division into items separated the incinerator furnaces from the other construction items, and thus facilitated the canvassing of bids and the selection of a contractor. With this and other considerations in mind, it was felt reasonable to allow monthly estimates amounting to not more than 70% of the value of the work properly performed at the time of the estimate. The

* *American Journal of Public Health*, April, 1926, Vol. 16, No. 4.

specifications were then developed in accordance with the following general outline.

Description.—The work to be done was described as the construction, testing, and placing in operation of the incinerating plant. The description included definitions of garbage and rubbish, the proportions of materials as collected, together with analytical data as a basis for guaranties and tests.

Divisions.—Bids were asked for under Divisions *A* and *B*. Both divisions required a complete incinerator plant, including the runway, building, furnace, chimney, sewers, sewage disposal plant, and all necessary appurtenances. Division *A* allowed the bidder considerable latitude in the design and arrangement of the plant. Division *B* required certain features considered desirable by the city and its engineer.

Drawings.—Each bidder was required to submit with his proposal, drawings in sufficient detail to show the construction of the various items comprising the furnace and appurtenances. Before starting work, the contractor was required to submit detail drawings for the entire work. These drawings were to be approved by the city officials and their engineer and the work was required to be built in conformity with the approved drawings.

Statements.—In addition to the usual statements covering the amount of power, the schedule of labor, the average temperature, the capacity, and the like, the specifications required a list of other plants then in operation to which the bidder referred as examples of his construction and which the city officials and their engineer could visit and investigate so as to satisfy themselves that the proposal could be carried out. It was stated that the city officials desired the information to clearly support the proposal of the bidder. Therefore, the statement relating to incinerators similar to the one proposed was required to be complete and to include features in which the design of the operating incinerator differed from the design of the proposed incinerator.

Guaranties.—All tests and failures were covered in much the same manner as in other similar specifications. The bidder was required to state the amount, if any, of additional fuel he considered necessary.

Workmanship and Materials.—Fifty-seven paragraphs were included on the workmanship and the quality of construction materials.

Items.—Sixty paragraphs were included to describe the five items of the work. Every effort was made to describe the items in such complete detail that the bid price would largely control in the selection. This was not entirely possible for the furnaces proper because of the variety of designs which might be submitted under the two divisions of the specifications.

General Conditions.—The usual general conditions, including thirty-four articles, covered all the general requirements of the work.

These specifications were accompanied by a set of four drawings indicating the location and general arrangement of the plant. Perhaps the special feature of these specifications was the requirement of references to operating incinerators similar to those proposed. As a matter of fact, the city officials and their engineer investigated, by actual visit, operating plants similar to those proposed by several of the bidders.

Present Practice as Regards Reduction

Most reduction plants have been built and developed by contractors having general agreements with cities for the service of garbage disposal. Garbage reduction works which are now in operation have been built by contractors at Boston, Mass., Pittsburgh, Pa., Baltimore, Md., Cincinnati, Ohio, Detroit, Mich., and other smaller cities. (See Table 1.) Plants taken over by cities, but originally built by contractors are those at Chicago, Ill., Philadelphia, Pa., Cleveland, Ohio, and Washington, D. C. The Cobwell plants at Los Angeles, Calif., and New York, N. Y., were designed and built by contractors, but they are not now in operation. The reduction works at Columbus, Ohio, and Rochester and Schenectady, N. Y., were built by the cities under more or less detailed plans and specifications. The plant at Syracuse, N. Y., was built by the manufacturer under a special agreement with the city and was later operated by him for a time.

Changes and improvements have been made in many of these projects. In some of the plants, odor problems and other operating defects have been encountered, and details for improvement have been designed. Much of this detailed experience is not available for general use in developing designs. Work of this character has been done at Detroit and Rochester and should develop interesting data. However, general design data are not on the whole as accessible as for sewage treatment and water purification works.

TABLE 1.—Administrative and Engineering Work in the Collection and Disposal of Garbage. Data Relating to Reduction Plants.

Plant.	Originally built by:	Now operated by:
Baltimore, Md.	Contractor	Contractor
Boston, Mass.	Contractor	Contractor
Chicago, Ill.	Contractor	City
Cincinnati, Ohio	Contractor	Contractor
Cleveland, Ohio	Contractor	City
Columbus, Ohio	City	City
Detroit, Mich.	Contractor	Contractor
Indianapolis, Ind.	Contractor	City
Philadelphia, Pa.	Contractor	City
Pittsburgh, Pa.	Contractor	Contractor
Rochester, N. Y.	City	City
Schenectady, N. Y.	City	City
Syracuse, N. Y.	City	Contractor
Washington, D. C.	Contractor	City

Except for the relatively few reduction works built under city plans and specifications, most of the plants in the larger cities of the United States have been built, as far as the municipality is concerned, under general specifications requiring certain standards of service. The contractor has been left free to design and detail the work proper. Many of the larger plants so built were placed in operation twenty to thirty years ago when their design was less advanced than at present. Many plants were developed by inventors and have since been renewed or rebuilt with more or less standard manufactured equipment. Consequently, the engineering work has been somewhat limited to the assembling into working units of a variety of readily purchasable

mechanical parts. This procedure has tended to develop a practice in the construction of reduction works akin to the purchase of boilers, engines, piping, generators, etc., in a power plant. Under conditions usually obtaining, it is perhaps likely that such purchases can be made more advantageously and the entire plant better assembled by a private organization than by a municipality. However, in both cases, the quality of engineering work will generally establish the success of the project.

Present Practice as Regards Hog Feeding

Garbage disposal by feeding to hogs is a development from the small cities under more or less informal agreements for collection and disposal. Few detailed plans and specifications have been prepared, with the exception of the Los Angeles Farm. The engineering work in the larger projects should include facilities for receiving and distributing the garbage, hog houses and pens, roadways and platforms, water supply, and sewerage, together with adequate methods for disposing of unconsumed garbage. Where proper design of such details is included in the construction of a hog farm, disposal by feeding to hogs is not as profitable as under a more informal procedure, except perhaps in large undertakings, but is less likely to create a nuisance.

Engineering Details

Table 2 represents a number of the items which indicate the engineering aspects of garbage disposal projects. This list is only general and approximate. Many details are of great importance, such as the design of furnace

TABLE 2.—Administrative and Engineering Work in the Collection and Disposal of Garbage. Engineering Items in Garbage Disposal Works.

Incinerators.	Reduction plants.	Hog farms.
Buildings and grounds	Buildings and grounds	Buildings and grounds
Unloading facilities	Unloading facilities	Unloading facilities
Storage arrangements	Storage arrangements	Distribution arrangements
Charging apparatus	Raw garbage conveyors	Roadways
Furnace arrangements:	Digesters	Hog houses
Structure	Dryers	Hog yards
Brickwork	Presses	Feeding platforms
Flues	Grease extractors	Fences
Grates and grate setting	Naphtha storage	Farm equipment
Ash and clinker removal	Grease refining	Final disposal of residues
Ventilation	Grease storage	Water supply
Stoking doors	Screens	Drainage
Tools	Crushers	Sewerage and sewage disposal
Combustion chamber	General conveyor system	Facilities for vaccination
Forced draft	Tankage storage	Facilities for farrowing
Dampers and draft control	Shipping facilities	Farm buildings
Chimney	Scrubbers	Silos
Approaches	Floor and floor drains	Auxiliary feed storage
Foundations	Office and laboratory	
Operating facilities	Chimney	
Recording instruments	Settling and skimming tanks	
Water supply	Boiler plant	
Sewerage and sewage disposal	Power plant	
	Foundations	
	Water supply	
	Sewerage and sewage disposal	
	Naphtha recovery plant	

bracing and fire-brick arches in the incinerators, and the materials used for piping, valves, and other metal structures subject to corrosion in reduction works. The general arrangements for the economical operation of reduction works are important, including the delivery and use of steam, the control of naphtha losses, and similar items affecting the over-all costs of operation.

Of special importance are the works needed for satisfactory odor control. This aspect relates to the broad considerations resulting in the selection of the method of disposal, and then to the details of odor control, such as the capacity of the incinerator grates, flues, and combustion chambers, and the confinement and treatment of odorous gases at reduction works.

It should be apparent that much operating experience is necessary to secure a sturdy and economical design. In some instances where the hazard of litigation over nuisance is forced into the situation, a careful weighing of the advantages of the different types is required, involving consideration of the first cost of plant construction, the cost of collection and haul as affected by disposal plant location, the net operating costs, and the cost and feasibility of adequate odor control.

Test and Routing Operating Data

Generally, the best design results from a familiarity with adequate, correct, long-time operating experience and data. As a basis for design, such operating data must be specific. Desirable information includes the following items:

A.—Characteristics of the Garbage.—

Quantities of Garbage:

- Seasonal variations,
- Daily quantities,
- Influence of climate.

Physical Characteristics of Garbage:

- Free water,
- Foreign material,
- Age at disposal plant.

Chemical Characteristics of Garbage:

- Moisture,
- Carbon,
- Ash,
- Volatile matter,
- Grease,
- Protein,
- Nitrogen,
- Potash,
- Phosphates.

B.—Incinerator Data.—(All related to the characteristics of the garbage):

Rates of Charging and Burning:

- Fresh and after storage.

Temperatures in:

- Furnace chamber,
- Combustion chamber,

Temperatures in (*Continued*):
- Flues,
- Chimney,
- Ash-pits.

Forced or Natural Draft:
- Pressures,
- Volumes,
- Temperatures.

Analyses of Products:
- Flue gases,
- Ashes and clinkers.

Economies of Additional Fuel.
Effect of Different Methods for Limiting Radiation Losses.
Structural Strains and Stresses as Related to Life and Design.
Preliminary Dewatering and Drying.
Combustion Chamber Operation in the Removal of Dust and Paper.

C.—Reduction Plant Data.—(All related to the characteristics of the garbage):

Effect of Age of Garbage on Recoveries.
Floors and Floor Drainage.
Rates of Operation of Driers:
- Water evaporated per hour at different temperatures,
- Coal required,
- Temperature and volume control,
- Vapor pressures and moisture absorption,
- Drier gas volumes,
- Analyses of drier gases,
- Losses of recoverable materials,
- Odor control of drier gases.

Design of Digesters:
- Operating data with different shapes,
- Steam economies,
- Volume and treatment of vent gases,
- Design of linings,
- Loading and discharge facilities,
- Value of stirring,
- Drainage features,
- Economical period of digestion.

Design of Percolators:
- Loading and discharge facilities,
- Inlet and distribution of steam,
- Drainage,
- Grease recovery,
- Condensers,
- Naphtha cycle.

Design of Presses:
- Condition of the garbage,
- Time of pressing,
- Feeding or charging the presses,
- Value of grease as compared with cost of pressing,
- Materials of construction,
- Drainage.

The foregoing list is suggestive rather than complete and a reasonable perspective must be taken regarding the precision of the data as affecting economies in the design. Attention should be given to the heat, water, and naphtha balances for the plant as a whole and for the several groups of equipment. Grease-skimming basins and the proper disposal of liquid residues are important. For some of the items too great precision is not warranted, but in the past the writer believes engineers have erred by not securing sufficiently precise records.

In the incinerator field, a great many data are available. Extensive tests, including analytical data, have been secured at Staten Island, New York, by Fetherston, at Milwaukee by the writer, and recently by other observers. Long-term operating records are not so complete and are few in number compared with the analytical data kept and published in the operation of water and sewerage works.

Less extensive operating data are available for reduction works, although the indications are that more are becoming public. Such information would clarify the design of driers, presses, and other equipment as regards operation, and would indicate plant balances in the economical use of coal, steam, water, naphtha, power, and labor.

There is need for more information and for a wider application and use of the experience already had. Work somewhat analogous to the extensive tests by municipalities and sanitary districts of various sewage treatment processes would be very helpful if applied in garbage disposal projects. Extensive sewage experiment stations have been operated in New York, Chicago, Philadelphia, Boston, Cleveland, Milwaukee, Akron, Gloversville, N. Y., Decatur, Ill., and elsewhere and in some of the State Universities. Similar tests of garbage disposal methods might best be undertaken at the larger plants somewhat as has been done at Detroit and Indianapolis. First, complete records would be established followed by special tests of several units and types of equipment. From time to time new kinds of machinery and equipment would be installed and tested. As compared with sewage works, relatively short tests would suffice as the processes involved are generally not biological. Such efforts should be characterized by accurate technical work, care in recording results, and the publication of the data and the conclusions.

The Patent Situation

Many of the various processes for garbage disposal are, or have been, covered by patents. The validity of these patents has never been tested as far as the writer knows. Some of them, such as those covering the Cobwell process, appear to be broad and general, whereas others relate to minor devices. There are several different "makes" of incinerator which seem to be quite similar in general design, but which are apparently carefully guarded by separate patents. Just what constitutes immunity from patent litigation in the garbage field is difficult to express. The present state of affairs, however, probably tends to cloud the general situation and to retard a more rapid development based on fundamental considerations.

Typical Steps in the Acquisition of a Garbage Plant

It is of interest to picture the procedure now followed by many communities in the acquisition of a garbage disposal plant. Action may be prompted by difficulties arising over some informal method of disposal, as by dumping or by feeding to hogs. Increased population and the encroachment of critical districts may require relocation of existing disposal facilities. Occasionally, the fault lies more with the collection than with the disposal.

When such conditions arise, city officials, sometimes with interested citizens, plan trips to other cities where they inspect garbage disposal plants in operation. Much information is furnished by representatives of manufacturers of different garbage disposal plants, and by other cities. The method of garbage disposal is selected, and proposals are invited so as to encourage competitive bidding. After the bids are received a comparison is made, necessarily on the basis of likely performance as well as on cost. A selection having been made, the contract is awarded and steps are taken to start construction.

At this point, injunction proceedings are frequently started to prevent the carrying out of the work. Sometimes objection is made to the proposed location. In other instances the method of taking bids and awarding the contract is questioned. Such proceedings are sometimes long drawn out and quite expensive. In many cities, unless fraudulent practice has been proven, the Court decision favors the municipality and the work proceeds.

When construction is completed tests are required. Occasionally, the delays noted have thrown the completion of construction into a new administration and difficulties arise over the final testing and acceptance of the work. Such difficulties are sometimes actual in that the plant itself can not meet the guaranties, and occasionally result from failure to give the plant a fair trial in operation.

If, however, the plant finally comes through these various stages it is more than likely to give favorable results in operation, particularly during the first few years. Many plants of faulty design are giving fairly good operating results, particularly where the collection service is good.

It seems to the writer that difficulties similar to those noted are less likely to occur if the project from the beginning to the end receives consistent, unhampered, and careful engineering work.

Limiting Procedures

The general practice in acquiring garbage disposal works has already been stated. There are limiting procedures which are worth describing.

One extreme includes the preparation of complete detail plans and specifications on which bids for construction can be secured. This means that the city itself would assume responsibility for the design and would proceed in the usual way to select a contractor for building the work. The detailed plans and specifications might be prepared by engineers acting for the city, or by experienced general manufacturers capable of making plans and specifications suitable for competitive bidding.

The other extreme comprises a contract for collection and disposal in which the work to be done is clearly stated and the methods are left to the successful contractor. The procedure at Kansas City, Mo., is along this line. The notice to bidders in Kansas City stated that:

"The work for which proposals are invited includes the complete and satisfactory collection and disposal of the garbage of Kansas City for a term of 10 years in a clean and sanitary manner, and the furnishing of all necessary plans, statements, equipment, buildings, plant, apparatus, sidings, land, labor, horses and appurtenances."

Throughout the specifications and contract, the work is consistently described as the service of collecting and disposing of garbage. As far as possible, this service was clearly and fully described. The following list of items from the specifications indicates the topics covered:

1.—Description of work to be done.
2.—Definition of garbage.
3.—Extent of garbage collections.
4.—Collection districts and headquarters.
5.—House treatment for garbage.
6.—Weighing garbage.
7.—Frequency of collections.
8.—Hours of garbage collection.
9.—Emergency collection service.
10.—Complaints.
11.—Regulations for sorting.
12.—Handling house receptacles.
13.—Maintenance and cleaning of garbage equipment.
14.—Painting collection units.
15.—Drainage of garbage.
16.—Transfer stations and equipment.
17.—Methods of garbage disposal.
18.—Disposal of residues.
19.—Inspection by the city.
20.—Workmanship and materials.
21.—Sanitary operation.

The specifications stated that the garbage could be disposed of by any method shown to have a satisfactory record of operation elsewhere. Among such methods, feeding to hogs, processes which recover saleable products, and incineration were included. The contractor was required to provide ample capacity and facilities so that the disposal plants could be operated in a clean and sanitary manner free from nuisance. Among such facilities the specifications included concrete floors, drains, water supply, housing, etc.

Particular attention was directed to the disposal of residues. The specifications stated that such residual products must be properly handled, if the disposal was to be complete, and listed the following:

(*a*) Unconsumed garbage at hog farms.
(*b*) Manure at hog farms.
(*c*) Ashes from incinerators.
(*d*) Sewage, including floor washings, scrubber wastes, grease liquors, etc.

(*e*) Gases from driers and incinerators.
(*f*) Tins, tailings, and rubbish from recovery processes.
(*g*) Tin cans and tinware.

A contract for this work has been awarded and in operation since the early summer of 1925. The bid price was $6.45 per ton for collection and $1.00 per ton for disposal. Under the contract temporary methods of disposal were permitted. A first-class collection service has been developed. From about 100 000 houses there have resulted about 25 complaints of non-collection service per day. Practically all complaints received before noon are handled by the emergency truck the same day. Complaints received during the afternoon are handled the following morning. Considering the background of garbage collection experience in Kansas City, this evidences good collection service.

Present Indicated Practice

Each project should, of course, be studied in the light of as much operating information and experience as possible. On this broad basis, the following steps generalize the best procedure indicated for a municipality in the acquisition of a garbage disposal plant:

a.—A choice should first be made between municipal and contract operation. If contract operation is selected, the specifications and contract should cover in detail the service to be rendered and the standards of operation required, as described for Kansas City. If municipal operation is selected, the procedure is somewhat different as indicated by the following items.

b.—The collection problem should first be studied and developed in sufficient detail to insure satisfactory service to householders, reasonable economy, and proper co-ordination of the collection and the disposal.

c.—A site or sites for disposal plants should be acquired which suit the collection service, and which can be clearly established as integral parts of a sound general plan for garbage collection and disposal for the community as a whole. This general plan must be sufficiently sound to survive injunction proceedings.

d.—The method of disposal best suited to local conditions and available sites should next be determined. The method selected should be defined as far as possible to promote a ready comparison of bids and yet to encourage competition.

e.—The City should then prepare detailed plans for the grounds, approaches, buildings, chimneys, and other appurtenances including the general arrangement of equipment, such as furnaces, boilers, driers, digesters, storage facilities, etc. These plans should be detailed to the greatest extent possible with due regard to maintenance of competition in the equipment field.

f.—The drawings should be accompanied with complete specifications in general accord with the arrangement and topics described on pages 803 and 804. These specifications should reasonably and justly protect both the city and the contractor by the elimination as far as possible of uncertainties and hazards.

g.—The specifications as outlined will require reference by equipment bidders to works now in operation similar to those offered in the proposal. Such operating equipment for the two or three more favorable proposals should be inspected and tested as operating units before selecting the most favorable bid. Such a procedure will temper the bids to a reasonable basis and will enable a fairer selection where the selection has to be made on a performance basis.

h.—Some plans detailing the equipment offered will be included with the proposal. Other construction drawings will be required of the contractor. All such plans should be carefully checked and approved by the engineer prior to and during construction.

i.—Construction should be carried out under careful engineering supervision and inspection. Some failures may be charged to poor construction work.

j.—After the plant is built, careful and fair operating tests should be made before acceptance. It is likely that recognition on the part of bidders that acceptance tests are to be made by competent, fair-minded engineers will promote safer and better bidding and will eliminate part of the contingent item in the bid.

k.—After the work is finally accepted, a satisfactory operating routine with adequate records should be established.

This procedure will be better followed if it is entrusted to experienced, disinterested engineers. Probably the best way to obtain results is through the development of a public opinion which understands the administrative and engineering aspects of the collection and disposal of garbage.

GARBAGE COLLECTION AND DISPOSAL, LANSING, MICHIGAN

By Edward D. Rich,* M. Am. Soc. C. E.

Prior to 1916 the garbage of Lansing, Mich., was collected by private contract involving a weekly charge to each household taking the service. In November, 1916, the citizens voted for a general collection to be paid for out of the annual tax levy.

Collection

With this change, the can system of collection was adopted, the City furnishing the cans. Motor trucks are used exclusively for collecting the cans which have a capacity of about 1 bushel and are tapered, being smallest at the bottom. The 2½ to 3-ton truck has been found to be the most satisfactory and economical although various sizes, from ordinary Fords to 3-ton trucks, are in use.

Calls are made daily at hotels and restaurants and weekly at residences. In making the collections, the can covers are not collected, as without the covers, the cans may be nested on the truck, thus consolidating the load. If this nesting is done carefully, no garbage is spilled on the ground or from the truck. The trucks are hosed after each trip.

It is believed that tank-wagon collection would be cheaper than the can system but considering the advantages of efficient and satisfactory service to the householder and also the condition of the garbage, the additional expense, if any, seems justifiable. The elimination of the disagreeable task to the housewife of washing garbage cans is a distinct service that is highly appreciated. The use of miscellaneous receptacles, which are so common if garbage is collected in tank wagons, results in deterioration of the garbage before collection, thus rendering it less suitable for feeding purposes.

The City has been unusually successful in obtaining separation of tin cans, glass, crockery, and other foreign materials from the garbage, accomplished largely through newspaper publicity. The people are asked occasionally not to jeopardize their own hogs by putting anything into the garbage that will injure them.

A charge of $1 per year is made by the City for each can in use, the money being kept in a separate fund for maintaining the supply of cans. The wisdom of making any direct charge to the householder is doubtful; the object sought in garbage collection is a clean city and any fee, however small, tends to deter some from taking the service. The cost of collection should be provided for in the tax budget and inspectors should be employed to see that every household avails itself of the service. The percentage of families who pay the can fee indicates that this charge has not had a very serious effect on the extension of collection over the entire city, but there seems to

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be no logical reason why the cost of maintenance of cans should not be paid out of the tax levy the same as the other items of collection cost.

The amounts received from can fees for four years are as follows:

1921	$ 8 403.95
1922	$11 058.90
1923	$11 675.14
1924	$12 884.25

It is stated that the cost of supplying the cans in use for the four years about equals the receipts from can fees. In some years there has been a deficit and in others a profit. A reserve supply of about 1 000 cans is kept on hand.

The cost of collection for four years is shown in Table 3.

TABLE 3.—Cost of Garbage Collection, Lansing, Mich., 1921-1924.

Year.	Cost.	Population.	Patrons served.	Population served.	Percentage of population served.	Cost.		
						Per capita cost to total population.	Per capita cost to population served.	Cost per patron.
1921	$36 245.54	61 242	7 600	31 540	51.4	$0.59	$1.15	$4.78
1922	41 869.03	63 852	8 500	35 275	55.3	0.66	1.18	4.93
1923	44 004.06	65 871	11 120	46 150	70.0	0.67	0.95	3.96
1924	44 363.07	67 091	11 800	48 970	73.0	0.66	0.91	3.76

These costs include labor, repairs to equipment, gasoline, salary of the superintendent, his office expenses, interest at 6% on the value of five trucks at $2 000 each, and depreciation on these trucks at 14 per cent. The population of the city since the 1920 Census has been estimated in proportion to its growth between 1910 and 1920. No exact record of the number of patrons is kept. The figures shown in Table 3 were estimated and are regarded as substantially correct. The 1920 Census gives a relation between the population and number of families in Lansing of 4.15 persons per family. This factor multiplied by the number of patrons gives the estimated population served as shown in Table 3. This method of estimating population served is perhaps not very accurate, but no better means is available.

From the marked increase of patrons in 1923 over 1922, it will be noted that the cost per patron, or per capita of population served, decreases as the percentage of population served increases.

Disposal

During 1916 a municipal piggery was established on a relatively small tract of land owned by the City. This property was not large enough to permit the keeping of sufficient hogs to consume all the garbage collected and nuisances resulted. During the latter part of 1921, a farm of 120 acres was purchased and the development of the present feeding plant begun. This

farm is 4½ miles from the center of the city, in a high-grade farming locality and in another county. The location of the piggery in another county and among good farms brought objections at once from the neighbors. An injunction was sought against the operation of the plant but the Court held that no injunction could be issued until a nuisance had been created. Later, Court action was again invoked and on February 16, 1925, an injunction was granted by the Circuit Judge of the county in which the pig farm is situated. This injunction was based on testimony that nuisances had been caused by foul odors from the pigs and more especially from the disposal of refuse from the feeding floors. The Court decided that the neighbors had been damaged in the enjoyment of their homes and that the value of their properties had been decreased. The case is now (1926) before the Supreme Court of Michigan for final review.* Awaiting the decision of the Supreme Court, the City continues to operate the plant with improved methods of disposal of the waste materials from the feeding floors.

The feeding plant proper occupies a tract of about 8 acres near the center of the farm and 700 or 800 ft. from the nearest road. The animals have a free run on this area. There are three feeding houses, each 33 by 100 ft., with concrete floors. These houses are divided longitudinally by a fence into two alleys, 16 by 100 ft. The collection trucks drive into these alleys and unload direct from the cans to the feeding floors before the hogs are admitted. After the garbage has been fully worked over by the pigs, the rejected refuse is thoroughly treated with lime. Formerly, this refuse was hauled in dump wagons to various fields on the farm and placed in one-wagon load piles. In the spring these piles were spread and plowed under as a fertilizer. So much odor nuisance was created by this operation that it was decided to distribute the material over the land by means of manure spreaders as fast as it accumulated. This seemed to be a marked improvement and less odors were created. After the injunction was granted, it was decided to institute further precautions, and the material is now buried in shallow trenches. The trenches are excavated with a road grader, the refuse is deposited in layers about 4 in. deep and covered by hand labor with about 8 in. of earth. It is expected that the land thus treated will be ready for crop use in about a year. This method of refuse disposal has eliminated all odors except pig smells which are not often noticed on the surrounding roads.

The garbage cans are thoroughly washed in a machine constructed especially for that purpose in a building near the feeding houses. This building also contains a room with steam pipes in the floor for thawing garbage in the cans so as to avoid damage in removing frozen garbage, and a steam plant for heating and for furnishing hot water for can washing and floor scrubbing.

Sleeping quarters for the hogs are provided in three buildings, 15 by 80 ft., two buildings, 15 by 100 ft., also about ten old colony-houses about 6 by 8 ft. The new sleeping buildings are divided into pens about 15 by 8 ft. There has been some trouble from sweating of the hogs with consequent danger of pneumonia. Decided improvement, however, has been obtained by building wooden box ventilators, 4 by 6 in. in size, that extend from about 2 ft. above

* For the decision of the Court in this case, see p. 851.

the floor through the roof. There is a ventilating shaft for each pen. The sleeping buildings are in pairs, with the space between the buildings paved with concrete. The entire yard around the buildings is scraped once each week to remove droppings.

Pigs are bought at a weight of about 120 lb. and kept until they weigh about 250 lb. They are kept in quarantine for 21 days during which time they are inoculated against hog cholera and pneumonia. The number of animals averages about 1 200 in winter to about 2 000 in summer.

The garbage received at the farm is not weighed and no accurate figures are obtainable to show how much is received during a year, but for 1924 the quantity is estimated at 19 350 tons. This figure was obtained by multiplying the total number of cans collected by the average weight of the contents of the cans.

The cost of disposal for the eight months, from May 1 to December 31, 1923, is as follows:

Receipts:		
Sale of hogs	$20 211.60	
Farm produce	905.84	
Increase in value of hogs on hand	13 500.00	
Total	$34 617.44	$34 617.44
Expenses:		
Hogs purchased	$15 652.37	
Pump and repairs	193.18	
Taxes	86.50	
Interest at 6% on $15 000 (farm)	600.00	
Insurance	32.37	
Lime	331.00	
Labor	7 782.17	
Repairs to buildings	122.92	
Total	$24 800.51	
Interest and Depreciation:		
Farm implements	$22.90	
Buildings	716.05	
Total	$25 539.46	$25 539.46
Profit for 8 months		$ 9 077.98

It is unfortunate that records are not available to show the financial results of operating the piggery over a period longer than eight months. It is believed, however, that these figures are not exceptional and that the profits during 1924 and in 1925 up to the time that burial of the refuse from the feeding floors was begun, have been similar to those shown for the last eight months of 1923.

GARBAGE DISPOSAL BY HOG FEEDING, LOS ANGELES, CALIFORNIA

By W. T. Knowlton,* M. Am. Soc. C. E.

From July, 1915, to September, 1921, all the garbage of Los Angeles, Calif., was delivered to a reduction plant owned and operated by a private company. In 1915, this company made a 10-year contract to dispose of the garbage, but, in 1921, it cancelled its contract as it claimed it was losing $15 000 per month.

In September, 1921, the City entered into a 10-year contract to sell its garbage for hog feeding. The garbage is collected by the City and shipped on steel gondola cars to the hog ranch of the Fontana Farms, which is in San Bernardino County, 55 miles from the loading station in the city. The City receives $0.60 per ton for the garbage after loading on the cars.

Before leaving the city, the garbage is classified as to whether it comes from hotels and restaurants or from residential districts. In the business district, garbage is collected every night, and in the apartment house districts and thickly populated residential districts collections are made three times per week. In the general residential districts, collections are made twice a week. The garbage collected from restaurants is used in the fattening of hogs, the other garbage being used for the smaller hogs which have not reached the fattening stage. Spur-tracks laid between the pens permit the garbage to be unloaded directly by cranes on to the pen floors, which are of concrete. To keep all areas where feeding takes place in a sanitary condition, all the spaces adjoining the railroad tracks and the drainage gutters are also paved with concrete.

The total area of the Fontana Farms is about 14 000 acres, of which, 320 acres are used for a hog ranch. Including the brooder group, there are about 44 000 hogs on the ranch, divided into different groups from the weaning stage to the fattening stage. The quantity of garbage fed to the hogs in the various groups depends on their development. After feeding, the waste is piled by plows and Fresno scrapers, and removed by wagons. This clean-up is then taken to a compost pit.

In 1921, an average of 216 tons of garbage was collected per day, the daily averages for the following years being 246 tons in 1922, 324 tons in 1923, 371 tons in 1924, and 371 tons in 1925. A part of this daily garbage is material that the hogs will not consume. In addition to this waste, the clean-up from the pastures and back pens is also collected. The total of this refuse is about 125 tons per day, of which 50 tons come from the feed floors, 40 tons from the back-pen clean-up (including therewith the hog manure), and 35 tons from the pasture clean-up.

The compost pit consists of two concrete bins, each 250 ft. long and 12 ft. deep. The side walls of each bin are 56 ft. apart at the bottom and 75

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ft. at the top. The concrete floor of each bin consists of alternate ridges and valleys equally spaced. The horizontal distance between each valley and ridge is 7 ft. and the vertical distance about 1 ft. Below each valley line is a concrete box-drain about 1 ft. square in cross-section, in which the drainage from the waste material is carried on a slope of 1 in 20 to an intercepting channel at the end of the bin. This intercepting channel discharges its flow into a sump from which the water is pumped into a sewer. A movable steel girder bridge, 75 ft. long, across the bin permits the waste material to be readily dumped into the compost pit, the bridge moving along rails laid at the top of the side of the pit.

When the feed-floor clean-up is dumped into the pit, it is covered with gypsum to prevent evaporation of the ammonia content. Peat was first used for such a cover, but later was abandoned, as it prevented moisture in the waste material from reaching the drains. Distillate is sprayed over the waste dump to prevent the breeding of flies. After being in the pit for four months, the material is removed by trucks, from an entrance at one end of the bin, and placed on a concrete floor, 120 ft. wide, that extends along the entire length of one side of the concrete bins, the floor being at the same level as the top of the bins. The clean-up from the pasture and back pens is spread on this floor to dry for a few days.

When placed in the compost pit, the waste material contains about 75% moisture, which is reduced to about 42% when it is removed to the drying floor. The compost product, together with the other clean-up after drying, is ground into a fine material which is hauled away to be used as a fertilizer in the orchards and vineyards of the Fontana property. An analysis of this ground-up product gave a water content of 5½ per cent.

The writer in interviewing those in charge of the hog ranch finds that they prefer to receive the garbage not more than a day old. Garbage is not wanted if it is more than 4 days old. The hogs are fed twice each day, the average daily meal being about 20 lb. per hog. To operate the hog ranch 3 trucks and 12 two-horse teams are required, together with a force of 110 men, mostly Mexicans.

THE DISPOSAL OF ORGANIC WASTE BY THE BECCARI SYSTEM AT SCARSDALE, NEW YORK

By Arthur Boniface,* Assoc. M. Am. Soc. C. E.

Every rapidly growing community, sooner or later, faces the problem of collecting and disposing of its household waste, in a sanitary, well-regulated, and economical way.

In 1922, the Village of Scarsdale, N. Y., having a population of 4 500, decided to provide for the collection and disposal of its waste by village forces. This decision was made with a view to correcting unsatisfactory conditions resulting from the then existing practice of doing such work by private contractors.

The village embraces about 7 sq. miles of territory and is almost wholly zoned for residential purposes, which has resulted in establishing a community of uniformly high-class homes. In March, 1923, a bond issue was authorized to provide funds for acquiring a suitable site and for the construction of a garbage disposal plant. Meanwhile, all the methods of garbage disposal were studied, particularly the system developed in Italy by Dr. Beccari.

The Beccari System first contemplated the reclamation of organic waste accumulated on farms, with a view not only to its salvage, but to an increase in its value and effectiveness as a fertilizer. Later, Dr. Beccari applied his system to the disposal of organic waste produced by municipalities, and met with complete success. From two cells built and operated in Italy in 1914, there are in use, or being built, in that country, about 1 800 cells.

It may be a minor consideration, but in communities like Scarsdale, the fact that a Beccari plant lends itself to architectural treatment is important, as much of the objection to other systems of disposal can be overcome. Having in mind the character of development in Scarsdale, and the fact that a disposal plant anywhere within the village confines might create a hardship to adjacent property owners, and faced with the necessity of a plant of some kind, the Beccari System had a peculiar appeal to those responsible for the disposal of that community's waste. The Beccari System was therefore chosen and an eight-cell plant built.

The Scarsdale Plant

It was anticipated that the Scarsdale plant would take care of an ultimate population of 8 000. It might be stated that, from records carefully kept, the production has been found to exceed all estimates and experiences, averaging almost 1 lb. per capita.

The plant commenced operation on December 3, 1923, and is now (January, 1926) disposing of the waste accumulated from about 6 000 people. It consists of four units, or eight cells, each cell having a capacity of 25 cu. yd.

* Village Engr., Scarsdale, N. Y.

Extension sheds were constructed on the sides and the tops of the cells, which, in addition to providing storage for the humus, furnish housing for the trucks and equipment. The charging is done through hatches on top of the cells and the discharging through doors built on the sides.

Each cell has a double floor, the bottom one draining to the sump. The upper floor consists of gratings, the openings in which equal 30% of the total floor area. At the four interior corners of the cell, ducts are built with openings at the height of the horizontal baffles or fillets which extend around the interior walls of the cell about 2 ft. apart. The object of these baffles is to assist in distributing the air through the mass, the air being admitted, in the first instance, through two port holes between the upper and the lower floors. Connected by a duct through the roof of the cell is a turret or tower in which are constructed five trays with openings at alternate ends. By this means, the escaping gases generated in the mass below, pass over each shelf, successively, before discharging into the open air.

Spread over the trays is an argillaceous material which, in turn, is covered with crystals of sulfate of iron. Its function is twofold: It fixes the products of the gases rising from the fermenting mass below and absorbs unpleasant odors that might otherwise escape and become a nuisance. The drip from the trays trickles back into the mass and undoubtedly assists in contributing to the high percentage of nitrogen and other chemical elements found in the residue after the cycle is completed. The cells function through the bacterial action of the micro-organisms developed in the fermenting mass.

Operation of Scarsdale Plant

The Scarsdale plant commenced operation by charging Cell No. 3 with about 45 cu. yd. of raw garbage. In all, about 17 cu. yd. in a period of 5 days were placed in this cell for its initial charge. On the third day after beginning operation, the characteristic drip became evident, and two days later this had increased very perceptibly. Also, on raising the trap-door above the cell, considerable heat could be felt, and vapor arose from the mass, showing that the first stage of putrid fermentation was in full progress.

At about the end of the third week the numerous fluidizing colonies prevalent in the early stages were observed to disappear, and a mould or fungus began to put in an appearance. There was no apparent diminution of heat even up to the time of discharging the mass which took place 45 days after the closing of the cell. The residue or humus from this first cell, and that of each subsequent cell as it was discharged, was stored in the shed adjacent, with the object of drying it and afterward grinding it for commercial purposes.

The humus as stored in a pile did not dry out as anticipated, and, further, at the foot of the pile, combined with the drip which continued, an appreciable hydro-carbon deposit was observed. It was thought that a longer period in the cell might reduce the moisture content of the humus, and, therefore, experiments were made leaving the mass undisturbed for periods varying from 50 to 65 days, but even with the longest period, no particular difference was noticed in the character of the humus produced, or its moisture content.

Efforts were made to dry the product in the open air on a platform erected just outside the plant, but this expedient was unsuccessful, and it became apparent that a properly designed drying shed must be constructed. Later, a shed, 100 ft. long and 12 ft. wide, was built on a farm in an adjoining township. It was constructed on sloping ground and permitted an arrangement, without great expense, whereby a lower floor accommodated the grinder. The humus from a cell was first dumped at one end of the shed and spread to a depth of about 2 ft.; then, at intervals of 2 days, it was turned over and gradually spread toward the other end of the shed above the grinder. At the end of 10 days it was sufficiently dry to be passed through the grinder, and sacked, after which it was sold as fertilizer. It was thus demonstrated that the humus could be made commercially valuable by natural processes. In using this means, however, there is danger, in built-up communities, of creating objections due to odors.

Control of Odors

There is another condition which must be met, namely, the control of odors during the discharge of the cells. It should be borne in mind that as the organic matter breaks down, the mass contracts to about two-thirds of its original volume. It, therefore, attains great density and compactness. An interesting fact, to which attention should be directed, is that this contraction is not accompanied by as great a reduction in the moisture content as might be expected. The effect of all this is the retention in the mass of quantities of ammoniacal and nitrogenous gases, which are naturally liberated when the cell is discharged. Under certain atmospheric conditions this is likely to be a serious matter.

In addition to the objectionable odor, the mass is in no condition to pass through a grinder, as by analysis it was found to contain about 45% moisture. The problem, therefore, is to break up the mass in the cell and induce the passage, if possible, of larger quantities of air. With this in mind, latticed wood cones, about 2 ft. square at the base and 4 ft. high, were built, the spacing of the bars being arranged to give each cone about 5½ sq. ft. of free openings.

The object was to increase by 200% the original free area provided by the floor gratings, and, therefore, seven of the cones have been placed in one of the cells to determine the effect on the mass. In addition to this experiment an attempt at drying the humus in the cell is now (1926) under way. For this purpose a fan, with a 25-h.p. motor, capable of handling 2 000 cu. ft. of air per min., has been installed. On the suction side of the fan, heating coils, connected with a 15-h.p. steam boiler, have been placed.

It is proposed to commence operation of this blower at the conclusion of the fermentation cycle, and maintain the temperature of the air passing through the mass approximately equal to that which the mass under normal conditions would develop, and to continue such operation until the mass is dry. This experiment will be watched closely and all the effects noted. Already the insertion of the wood cones in the cell has resulted in advancing the various phases of the fermentation cycle by several days.

Conclusions

As this paper must be considered more in the light of a progress report, definite conclusions will not be stated. The results to date, however, can be summarized as follows:

The plant was inexpensive to construct on account of its simplicity, and its operation has required only labor of the ordinary class.

Except for $6 for the purchase of the sulfate of iron for charging the turrets, the plant has cost nothing for maintenance.

The analysis of the humus shows the following average content:

	Percentage.
Nitrogen	3.0
Phosphoric acid	3.5
Potash	0.6
Moisture	45.0

Experiments with the humus in vegetable gardens and in flower beds has definitely shown its agricultural value.

The arrangement for absorbing odors from the cells during the fermentation process is adequate and entirely successful.

Some means must be devised to reduce the moisture content of the humus while it is still in the cell in order to avoid objectionable odors during its discharge and to facilitate its preparation for the market.

Although the moist humus, as taken from the cell, emits a characteristic odor, no trace of odor of any kind can be detected when the humus has dried.

It has been conclusively demonstrated that the system is peculiarly adaptable to communities like Scarsdale, and other high-class residential developments.

Also, with a properly designed drying unit, and perhaps an air washer, there is no limit to its applicability for the disposal of organic wastes.

In conclusion, there is no doubt that a new means of disposal of organic waste has been devised; that it is one of the most scientific and sanitary methods yet invented; and that, with the perfection of one or two minor details of operation, it will prove of great value to those charged with the responsibility of the disposal of community waste in a safe and economical manner.

HIGH-TEMPERATURE INCINERATION AT TORONTO, ONTARIO, CANADA

By J. A. Burnett,* Esq.

This paper is intended to outline briefly the principles of refuse disposal at Toronto, Ont., Canada, by the method of the high-temperature destructor.

Prior to the re-organization of the civic departments in 1913, most of the city refuse was dumped, the remainder representing about 75 tons per day, was destroyed by a low-temperature natural-draft incinerator of somewhat antique design, which was built about 1890.

Both methods were entirely unsatisfactory from a sanitary viewpoint. The old incinerator was rebuilt and modified to improve operating conditions, and after careful studies and surveys, the Street Cleaning Department, jointly with the Medical Health Department, finally recommended disposal by a high-temperature destructor. This recommendation was endorsed by the City Council, which authorized an expenditure of $1 000 000 for the refuse disposal program.

Early in 1917, a plant with a guaranteed capacity of 180 tons in 24 hours, was completed and placed in operation. This plant, which was built to the Department's specifications, consists of three, high-temperature, "Sterling", 4-cell furnace units, with combustion chambers, air heaters or regenerators, connecting flues, etc., and all appurtenances, including a radial brick chimney. The performance records of this plant have been very satisfactory during the eight years of its operation. Approximately 50 000 tons of mixed refuse is disposed of annually, which is a furnace capacity of 4½ tons per hour (the guaranteed capacity being 2¼ tons per furnace-hour).

The plant is in close proximity to at least five public institutions and is quite near a good class residential district. It is situated on low-lying ground with banks on three sides; the vehicles delivering material enter and leave the plant over bridges almost level. There are no complaints on record with respect to the operation of the plant.

During 1923, an additional destructor plant was recommended by the Department, to replace the old crematory, and also to provide for the disposal of considerable refuse that was being dumped. Specifications were prepared for a system similar to that of the former destructor, with the improvements incorporated that were found to be of benefit in the experience of operation and maintenance. The new plant, which is known as the Wellington Destructor, was placed in operation early in 1925, and finally accepted on May 1, 1925. It consists of four high-temperature, "Sterling", 4-cell furnace units, with combustion chambers, air heaters, flues, etc., and all appurtenances, including two

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radial brick chimneys, complete with self-supporting fire-brick lining, which extends the full height of the chimneys. Each chimney is 175 ft. high, with an internal diameter of 90 in. The guaranteed capacity of this plant is 400 tons per 24 hours.

The building of each plant was designed by the City Architect's Department, which also supervised the construction. The work was done by local contractors, but all engineering and supervision of the plant equipment was under the jurisdiction of the Engineering Division, Department of Street Cleaning. Both plants are of fire-proof construction throughout. The cost of the former, or Don Plant, was $225 000, and that of the Wellington Plant, $550 000, exclusive of land in both cases.

The furnaces are of the continuous-grate regenerative type, the cells being divided by low-set air-cooled castings which render it possible to fire and clean the grates separately. There are no moving parts inside the furnaces; each furnace unit has four cells, and a charging container for each cell; common combustion chamber, air heater, and by-pass flue. The material from the charging container drops on a drying hearth at the back of the grates, afterward being moved forward and spread uniformly on the grates, where incineration takes place. The gases pass over the grates into the combustion or settling chamber. At the back of this chamber a fire-brick arch splits the gases, a part of which pass into the regenerating chamber, and through the air-heater tubes, the remainder entering the by-pass flue, thence through the connecting or main flues into the chimney. The cool gases after passing through the air heater also discharge into the main flue.

Substantial steel buckstays are set at frequent intervals, securely tied top and bottom with rods to resist expansion stresses. All skewbacks are supported by heavy steel angles extending the full length of the various arches. The fire-brick lining of the furnaces, combustion chambers, and flues, is not less than 9 in. thick, and the outside walls are 1½ brick thick, except the connecting and main flues, which are only 1 brick in thickness. An air space of at least ½ in. between the lining and walls is maintained throughout.

The general arrangement of the flues is such that any desired battery of three units may be operated with either chimney, so that the plant may be operated at not less than 75% capacity at all times. The dimensions of the combustion chambers are such as to permit a rolling action of the gases. The approximate flue velocity is 20 to 25 ft. per sec.

The quality of the material delivered to the plants, as in every municipality, varies to some extent. The refuse delivered to the Wellington Plant is more uniform than that hauled to the Don Plant, which on certain days contains a greater proportion of rubbish. However, the average material for both plants is about 55% moisture, 34% combustible, and 11% residue. Burning this grade of material, an average combustion chamber temperature of 1 600 to 1 800° Fahr. is maintained at the rate of 4 to 5 tons per furnace-hour. The refuse consists of a mixture of rubbish and garbage, the approximate proportions being 60% garbage and 40% rubbish, by weight; no ashes or incombustible waste is hauled to the plants.

The rate of burning at the new plant is greater than that at the old one, on account of the increased width of the drying hearth, which renders the total grate area effective for burning, although the Don furnaces have an equal grate area, 25% of which at least functions as a drying hearth. At both plants, all material entering as well as the residue is weighed. The reduction by weight is 89 per cent.

Electric motor-driven blowers furnish the forced draft. Those installed at the Wellington Plant are equipped with dual inlets. When the furnaces are burning, the air is drawn from the charging floor level, but when the grates are being cleaned, the smoke is drawn from the ash-run and returned into the furnace.

The fans at the Don Plant furnish 7 500 cu. ft. per min. at a static pressure of 6-in. water gauge, whereas those at the Wellington Plant furnish 10 000 cu. ft. per min. at a static pressure of 4-in. water gauge. The estimated quantity of air required is 416 per lb. of refuse.

The pre-heated forced draft is considered to be a marked improvement in the development of the high-temperature destructor. It is understood that the Don Destructor was the first plant in American practice in which the gases at final combustion-chamber temperature were passed through the air-heater tubes. The method, however, has been successful to the extent that, after seven years of continuous operation, the bottom tube plates of one heater only have been replaced. Cast-iron tubes, 5 in. by $\frac{5}{16}$ in. thick, were installed for the Don heaters. Seamless steel tubes were furnished for the Wellington Plant, which have the advantage of transferring heat more rapidly, and also of reducing materially the dead load on the bottom tube plates. The durability of the steel tubes as against those of cast iron, due to the action of the gases, is problematical.

Pre-heated forced draft to a final temperature of 300° Fahr. (obtained normally), is a great factor in efficient burning. At this final temperature, the theoretical heat value put into the air, which is equivalent to 40 to 50 lb. of coal per ton of refuse, is claimed by some authorities to be a great advantage. However, from a practical operating viewpoint, the air is more efficiently distributed throughout the fuel bed on the grates, than it would be if coal were supplied to furnish the same heat. Also, the increase in volume, which is approximately $1\frac{1}{2}$ times the volume at initial temperature, aids combustion by a better distribution throughout the refuse for the supply of oxygen, thereby requiring less excess air than would be the case without pre-heating. The capacity when operating under natural draft has been found to be 60% of that under forced draft conditions.

The furnaces of both plants are charged by hand, each charging container having a capacity of approximately $\frac{3}{4}$ cu. yd. With the system of light charges at frequent intervals, the best burning results are obtained, namely, incandescent fuel bed, high rate of burning, and uniform temperatures.

Grates are normally cleaned three times each shift of 8 hours. The charging container and furnace doors are operated by compressed air rams.

The labor requirements are as follows: 1 foreman of plant; 1 leading stoker per unit for each shift; 3 charging men; 2 stokers; and 1 ash-run man. Table 4 shows briefly the statistics for 1925.

TABLE 4.—Operating Statistics of Incinerating Plants, Toronto, Ont., Canada, for 1925.

Plant.	Total experimental laboratory report, etc.	Loads, in pounds.	Tonnage.	Man-hours.	Man-hours per ton.	Residue, in tons.	Percentage of residue.
Don.....	$75 648	44 940	50 159.708	76 746	1.55	5 265	10.5
Wellington...	70 448	53 286	63 351.430	83 806	1.33	6 983	11.

The total cost of operation including labor, repairs, materials, and supplies, less carrying charges, for the Don Destructor is $1.50 per ton and for the Wellington Destructor, $1.11 per ton. No repairs or renewals were made for the Wellington Plant during 1925. Daily records of the complete operation of these plants have been made, dating back to July 1, 1917, for the Don Plant, and to January 1, 1925, for the Wellington Plant. Combustion-chamber temperatures are recorded continuously by electric recording pyrometers.

THE COBWELL SYSTEM OF GARBAGE REDUCTION AND SOME PHASES OF ITS OPERATION AT ROCHESTER, NEW YORK

By John V. Lewis,* Esq.

Historical

The idea of the "Cobwell System" of garbage reduction was conceived in 1913 by Mr. Raymond Wells, who spent the next two years in the development of this system, in collaboration with the C. O. Bartlett and Snow Company, of Cleveland, Ohio. As a result of their experiments and endeavors, the first practical Cobwell Plant commenced operation at San Francisco, Calif., early in 1915. This was a small 2-unit plant, with a capacity of 10 tons per day, for handling the garbage of the Panama-Pacific International Exposition.

About the middle of March, 1915, the Pacific Reduction Company opened the first large Cobwell Plant at Los Angeles, Calif. This plant contained 48 Cobwell units with a total daily capacity of 240 tons, and its operation was continued over a period of about six years, the plant at times handling as much as 300 tons per day.

Subsequently, four Cobwell reducers were installed at the plant of the New Bedford Extractor Company, New Bedford, Mass., and their use continued until the plant was destroyed by fire in June, 1923.

The largest Cobwell Plant installed to date was placed in operation July, 1917, by the Metropolitan By-Products Company on Lower Staten Island, Borough of Richmond, New York, N. Y. This huge plant was provided with 196 Cobwell units and had a capacity of about 1 300 tons per day. It was constructed to supplant the old Barren Island Plant. However, its operation was discontinued after a period of about fifteen months and has never been resumed.

During the next four years only two Cobwell Plants were built, one at Syracuse, N. Y., and the other at Rochester, N. Y. The Syracuse plant was a 12-unit installation, capable of handling 60 tons per day. In May, 1923, however, preliminary dehydrators were added, thereby changing the process to the modified Cobwell System and increasing its original capacity to 85 tons per day.

The Rochester Plant consists of 36 Cobwell units and has a maximum rated capacity of 180 tons per day. It has now been in continuous operation for a period of more than four years, in strict adherence to the original principles of the Cobwell System. Excluding the plant of the Pacific Reduction Company at Los Angeles, the Rochester Plant has to its credit the longest period of unmodified operation of the several plants thus far constructed and is now the only straight Cobwell Plant in use.

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During 1925, operation of a modified Cobwell Plant was commenced at Schenectady, N. Y. This plant contains only four Cobwell reducers but, with the dehydrator equipment, it is capable of handling 90 tons per 24-hour day. A plant of similar type, with a daily capacity of about 400 tons, has recently been proposed for Cleveland, Ohio.

This summary and Table 5 outline briefly the history of the development and installation of the "Cobwell and Modified Cobwell Systems" over the short period of ten years. Compared with the older systems of garbage reduction still in use after a period of forty years, it represents a distinct advance from the sanitary standpoint. Engineers may differ as to its economic operation or advantages, in comparison with other sanitary methods of garbage disposal, but this fact remains: The plants which are both in and out of operation stand as milestones along the path of progress in the field of municipal sanitation and the "Cobwell" or a similar type of system will supplant the older processes in those cities which have always looked with favor on reduction as a means of garbage disposal and have used it with more or less apparent success.

The Cobwell System

The most important part of the Cobwell System equipment is the so-called "reducer". This machine is very simple in design, consisting of a flat cylindrical steel shell about 10 ft. in diameter and 4 ft. high. A steam jacket is provided under the bottom and around the lower two-thirds of the circumference of the shell. A set of cast-steel plow arms, mounted on a vertical shaft with top and bottom bearings at the center of shell, are rotated close to the inner bottom sheet of the reducer. These arms slide under the mass of garbage, imparting to it an undulatory motion.

The mechanism for driving the plow-arm shaft at 10 rev. per min. is mounted on the flat top of the reducer. A charging door; a peep-glass for observing the action within the reducer; and an outlet for water and solvent vapors are spaced about on this same top. The discharge door is set into the reducer shell at the low point of its circumference, and several perforated drain-boxes project upward into the bottom of the reducer. These boxes serve not only as outlets for removing the grease extracted from the garbage, but also as inlets for the volatile solvent and live steam introduced at various periods in the reduction cycle. Each reducer is equipped with an individual steam trap and the necessary steam, solvent, and drain-valves, drain sight glass, and piping, and is supported on a heavy structural iron frame.

In other respects, the layout of a Cobwell plant does not differ greatly from that of the older systems. The usual types of conveying equipment for handling the green garbage and the rough and finished tankage are provided. There are electric prime movers and power-transmission equipment; surface condensers and condensate, solvent, and grease pumps; and steam stills, and the various separator and storage tanks for water, solvent, and grease. There is a mill room, equipped with crushing and screening machinery, for converting the rough tankage into a granular fertilizer base.

TABLE 5.—Historical Data on Garbage Disposal Plants Utilizing the Cobwell System of Reduction.

Plant located at:	Type.	No. of units.	Rated capacity, in tons per day.	Operated by:	Operation begun:	Period of operation.	Remarks:
San Francisco, Calif.....	Straight	2	10	C. O. Bartlett & Snow Co....	1915	10 months	Panama-Pacific Exposition.
Los Angeles, Calif.....	Straight	48	240	Pacific Reduction Co.....	1915	6 years	Operation discontinued in 1921.
New Bedford, Mass.....	Straight Modified	4 4*	20 ...	New Bedford Extractor Co.. City of New Bedford	1915	8 years	Destroyed by fire in 1923.
New York, N. Y.....	Straight	199	1 300	Metropolitan By-Products Co	1917	15 months	Operation discontinued in 1918.
Syracuse, N. Y.....	Straight Modified	12 12*	60 85	City of Syracuse..... Cobwell Reduction Co., Inc..	1921 1923	5 years	In continuous operation.
Rochester, N. Y.....	Straight	36	180	City of Rochester.....	1921	4 years	In continuous operation.
Schenectady, N. Y.....	Modified	4*	90	City of Schenectady.....	1925	9 months	In continuous operation.
Cleveland, Ohio.....	Modified	....	400	City of Cleveland	....	.....	Proposed installation.

*With preliminary dehydrators.

The straight Cobwell process in itself is carried on to its conclusion as a single continuous operation in the closed reducer from which odors do not escape. Furthermore, after a reducer has been charged, the several operations utilized in the older processes and involving the use of hand labor and the movement of the garbage to and from different kinds of equipment are not required. The reduction cycle consists of five steps as follows:

(1) Loading or charging of the garbage into the reducer.
(2) Dehydration or cooking, for the removal of free and combined moisture.
(3) Washing or extracting of the grease and oils.
(4) Drying or steaming of the tankage.
(5) Dumping or discharging of the rough tankage from the reducer.

The complete reduction cycle consumes from 18 to 24 hours, dependent on the character and condition of the garbage. The so-called cooking period requires from 14 to 17 hours and the washing period, with three washes, from 3 to 4 hours. Steaming of the tankage ordinarily occupies 1 hour. The loading and dumping periods are comparatively short, of 10 to 20-min. duration only.

Perhaps the most interesting feature of the whole process is that of the temperature produced during the reduction of the garbage mass. Entirely submerged in a bath of petroleum naptha, which acts not only as a heat transfer medium, but also a preventive of bacterial growth and hydrolysis, the garbage gives up its moisture content and disintegrates at a temperature of about 208° Fahr. This low temperature is not exceeded at any time in the reduction cycle and therein lies the reason for the non-production of obnoxious odors. The so-called "digestion" and digestion odors common to the older systems are not produced.

Aside from the important sanitary factors, the Cobwell System provides a higher recovery and larger quantity of valuable by-products than the older systems and a superior quality of the tankage as well.

Garbage Collection and Reduction at Rochester, N. Y.

From the best available information, it appears that the collection and disposal of the garbage of Rochester, as a municipal function, was undertaken about 1880. During the next thirty-seven years, this work was done mostly under contract with private collectors and contractors. In the first twenty-six years of this period, the collected material was hauled outside the city limits and disposed of by dumping or burying. When, in 1904, due to numerous complaints, it became evident that some other and more sanitary method of disposal must be adopted by the City, Edwin A. Fisher, M. Am. Soc. C. E., then City Engineer, was empowered to make a special survey and study of the situation, with recommendations for a better system of collection and disposal than was then in use. Mr. Fisher completed his report in September, 1906, and, as a result of his recommendations, the City awarded a 10-year contract for the collection and disposal of garbage, effective from January 1, 1907, to the Genesee Reduction Company. This Company proceeded to acquire a site on the west bank of the Genesee River between the Upper and

Lower Falls. The location chosen was within 500 ft. of State Street, an important business street, and within ¾ mile of the geographical and business center of the city. The narrow portion of the river bank at this point, on which the plant was built, lies about 150 ft. below the level of the city streets bordering on the river gorge and adjacent thereto. It is interesting to note that this same site is also occupied by the new "Cobwell System" plant which was erected in 1920-21. From the standpoint of the collection and haulage of garbage, it is an excellent location but, at the same time, a critical one with respect to possible nuisance caused by obnoxious odors which are inherent with the older systems of garbage reduction.

The Genesee Reduction Company began the operation of its plant early in June, 1907, using a modified Arnold, or so-called "Beaston", Process. At the conclusion of the 10-year period, the City purchased the collection equipment and the disposal plant from the contractor and has proceeded to carry on its own garbage collection and disposal systems since that time under the direction of the Department of Public Works. The modified Arnold Plant was operated continuously up to the date of the opening of the Cobwell Plant on October 15, 1921. Thereafter, its use was discontinued, except to utilize the digesters for grease-storage purposes and a few of the buildings for a repair shop and stock and dry storage rooms supplemental to the new plant.

The writer can only surmise the conditions and reasons which brought about the adoption of the new system of disposal. Although formal complaints against the operation of the old plant might have been few, there is evidence that verbal objections were many and that considerable money and effort were expended in an attempt to overcome the obnoxious odors and conditions attendant on its operation. Furthermore, the original equipment had reached the point where repairs were becoming more and more frequent and costly, and the end of its useful life was in sight.

In 1912-13, the City had constructed and commenced the operation of a Rubbish Salvage and Disposal Plant adjacent to the Reduction Plant. It originally contained a single DeCarie incinerator, but the capacity having been exceeded, the City proceeded in 1916-17 to install two new units, each with a capacity for burning 50 tons of rubbish per 24 hours. In addition to the salvage of saleable materials, the plant generated a considerable quantity of saturated steam which was purchased by the Rochester Railway and Light Company and delivered into its commercial heating mains. On the beginning of operation of the Cobwell Plant, the City diverted the supply of steam there, making it auxiliary to the supply of high temperature steam furnished to the new plant by the Power Company. This procedure has served to reduce the actual cost of the total quantity of steam required for operation of the new garbage plant in that each pound produced at the Rubbish Disposal Plant represents a saving of an equal amount which must be purchased in the absence of such incineration units and waste heat boilers or failure to operate them.

The Cobwell Plant and Some Phases of Its Operation

As has been heretofore mentioned, the new plant was opened in October, 1921. When, at the beginning of 1924, Harold W. Baker, Assoc. M. Am. Soc.

C. E., became Commissioner of Public Works, and the writer was appointed Sanitary Engineer in charge of the operation of all refuse disposal plants, the Cobwell Reduction Plant was found to be badly in need of repairs. Its condition, after only 27 months of operation, was such as to cause excessive consumption of steam, solvent, and condenser-cooling water and to endanger the safety of not only the plant and its personnel, but of the adjoining properties as well. Furthermore, records of operation and data, particularly with reference to the quantity of garbage handled and the quantity and quality of by-products produced, were very meager and unsatisfactory. No means had been provided at either the old or the new plant for weighing the quantity of garbage delivered daily for treatment, and a suitable platform scale installation was one of the first additions under the new régime. Replacement parts were procured and the repair work was prosecuted as rapidly as possible. A suitable system of record keeping was inaugurated and the operation of vital plant equipment checked by means of the installation of steam flow meters and pressure, vacuum, and temperature instruments of the various indicating, recording, and integrating types. Almost immediately, a new spirit of co-operation and endeavor began and within the short period of six months, the equipment had been restored and the plant was again operating as its builders intended it to perform. Thereafter, everything possible was done to achieve satisfactory results in the daily routine use of the process and, at the same time, reduce the operating costs at the plant. It is beyond the scope of this paper to discuss in detail the experiments and developments carried out during 1924 and 1925, a study which has furnished much of interest from the chemical, mechanical, and economic standpoint. The writer ventures, however, to present for the information of interested sanitary engineers and municipal officials, a summary of cost of operation during this period. These data are given in Tables 6 to 9, inclusive.

TABLE 6.—Cost of Garbage Disposal by Cobwell Process at Rochester, New York. Capital Charges.

Item.	Cost.	Interest and depreciation.*
Land	$47 140	$1 885
Buildings	138 090	82 130
Equipment	568 960	
Total capital charge		$84 015
Capital cost per ton of garbage treated		1924†..33 200 tons..$2.50 per ton. 1925 ..34 700 tons..$2.40 per ton.

* Interest charges are variable from 4 to 5 per cent. Depreciation charges are estimated at 2% on buildings and 8% on equipment.

† The quantity of garbage treated during the first three months of the 1924 period is estimated.

Table 6 sets forth the capital charges on the land, buildings, and equipment which constitute the Cobwell Plant. These charges have been reduced

to a tonnage basis for the years 1924 and 1925. In Table 7, the several major items which make up the operating costs of the plant are indicated over the same period of time and for the respective yearly tonnage. Table 8 presents the quantity of saleable materials produced, together with the revenues obtained therefrom in 1924 and 1925, and an estimate of the probable revenues for 1926. In Table 9, the total cost for the collection and disposal of garbage in 1924 and 1925 is summarized. These tables are concise, and the writer desires to explain and amplify certain of the figures given therein.

TABLE 7.—Cost of Garbage Disposal by Cobwell Process at Rochester, New York. Operating Costs.

Item.	Period.			
	1924		1925.	
	Amount.	Percentage of total.	Amount.	Percentage of total.
Salaries and labor	$74 225	25.0	$72 285	32.0
Steam	84 730	29.0	67 050	29.0
Power and light	9 230	3.0	9 890	4.5
Solvent	43 670	15.0	23 550	10.0
Equipment and repairs*	40 115	14.0	20 640	9.0
Supplies and miscellaneous	31 050	11.0	25 335	11.0
Royalties	9 125	3.0	10 100	4.5
Total operating costs	$292 175	100.0	$228 850	100.0
Operating cost per ton of garbage treated			1924..$8.80 1925..$6.60	

* Includes cost of all new equipment and extraordinary repairs.

In the matter of capital charges, that for disposal is about the same in both 1924 and 1925, due to the slight increase in the tonnage treated in the latter year. No capital charges for collection are indicated, because the actual depreciation on the present equipment has been almost wholly absorbed. Such capital charges as were justified have been included as an arbitrary part of the collection operating costs during 1924 and 1925. In the operating costs for disposal, the largest items are those for labor, steam, solvent, equipment and repairs, and supplies. It will be noted that the cost for both steam and solvent, in 1925, was $18 000 to $20 000 less than that for 1924. The reduction in each case was due, largely, to the repairs made and the methods of operation introduced during 1924. At the same time, there was a substantial increase in the quantity of steam produced during 1925, at the Rubbish Disposal Plant. The cost of all extraordinary repairs that were made and of the new equipment installed during the two years, a good part of which might more properly be termed a capital charge, is included in the operating costs for the respective years. Altogether, about $65 000 has been expended for repairs and new and improved equipment during the two-year period. The low operating cost for collection of garbage is due, principally, to the central

location of the disposal plant with respect to collection routes and to the kind of labor and equipment which can be utilized under such circumstances. The average length of haul does not exceed 3 miles, and horse-drawn equipment is used almost exclusively.

TABLE 8.—Cost of Garbage Disposal by Cobwell Process at Rochester, New York. Revenues.

Item.	Period.		
	1924.	1925.	1926.
Quantity of grease produced, total pounds	2 189 900	2 042 900	2 300 000*
Garbage treated, in pounds per ton	66.0	58.9	65.0*
Grease revenue:			
Contract price per 100 lb	$5.40†	$6.25†	$7.00†
Average net price per 100 lb	$5.29	$6.00	$6.90*
Revenue per ton of garbage treated	$3.50	$3.55	$4.50*
Quantity of tankage produced, total tons	6 050	6 500	6 750
Garbage treated, in pounds per ton	365	375	380*
Tankage revenue:			
Contract price per ton	$8.10‡	$8.10‡	$8.10*
Average net price per ton	$8.18	$7.90	.....
Revenue per ton of garbage treated	$1.50	$1.50	$1.50*
Total revenue per ton of garbage treated	$5.00	$5.05	$6.00*

* Estimated.

† Based on 97% saponifiable oils.

‡ Based on ammonia, 3.45%, $2.02 per unit; bone phosphate lime, 4.70%, $0.10 per unit; and potash, 1.10%, $0.60 per unit.

TABLE 9.—Summary of Cost of Garbage Collection and Disposal at Rochester, New York.

Item.	1924	1925
Cost of Collection: *		
Capital charge	$0.00	$0.00
Operation	3.35	3.45
Total	$3.35	$3.45
Cost of Disposal: *		
Capital charge	$2.50	$2.40
Operation	8.80	6.60
Total	$11.30	$9.00
Cost of Collection and Disposal: *		
Total	$14.65	$12.45
Revenues	5.00	5.05
Net	$8.65	$7.40

* Per ton of garbage.

The revenues obtained from the sale of grease and tankage show that there was a fair market for these materials during 1924 and 1925. There are indi-

cations that the same conditions will continue through 1926 and perhaps for some time to come.

The figures given in the summary, Table 9, for 1925 are considered representative of those costs which may reasonably be expected at Rochester with the proper operation of the present collection and disposal systems. Deducting the revenues from the total cost for collection and disposal of garbage, the net cost per ton amounted to $7.40. If the cost of collection and the capital charges against the disposal plant are omitted, the operating cost for disposal alone is $1.55 per ton of garbage treated. There is every indication that this cost will be reduced during 1926 to $0.75 per ton, mainly through closer control of the process and the installation of new equipment.

In conclusion, the writer can only state that the operation of the Cobwell Process during 1924 and 1925 has shown it to be a satisfactory means of garbage disposal, particularly from the sanitary standpoint. The desired results have been accomplished at a cost which is neither excessive nor prohibitive under the local conditions which are peculiar to Rochester and with a definite advantage over the methods which were previously used there.

DISCUSSION

WALTER A. SPERRY,* ESQ. (by letter).—Prior to 1908 the garbage of Grand Rapids, Mich., was burned in an incinerator. At present (January, 1926), the incinerator is used to burn trash and the carcasses of animals. In 1924-25, 6 315 loads of refuse, and 8 700 odd animals, and birds, were destroyed. It is possible that a boiler may be operated in conjunction with this plant to provide heat during the winter to a group of utility buildings, shops, and garages for the use of the Service Department.

Beginning with 1908, the City of Grand Rapids adopted the plan of contracting with private parties for the disposal of its garbage by hog feeding. From 1908 to 1925 three or four companies have successively taken over this business, and at present (1926) one firm has a contract that will expire in two years. This method of disposal has been quite satisfactory, and, no doubt, will be continued. The general provisions of the contract, as it is now written, are as follows.

The City collects the garbage and delivers it to a central loading point, where it is transferred from the collection tanks to steel cars of the gondola type, eight of which are owned by the City. The refuse is then transported to the pig-feeding farms, about 45 miles northwest of the city. For this the City receives payment at the rate of $50 per month for the total garbage received, and, in addition, receives from the contractor $150 per year toward the cost of maintaining the railroad equipment. In the latest contract the City agrees to contribute toward the freight at the rate of $12 per car, the freight having been adjusted on a car basis in place of a tonnage basis, so that there are no restrictions on loading cars to capacity, the contractor agreeing to dispose of all garbage regardless of its amount. The contractor further agrees to keep the cars in a clean and sanitary condition, and the City agrees to protect the contractor's rights to all garbage accumulating within the city limits. Finally, the City is protected by a bond of $10 000 for the faithful performance of the contract.

The City has no control over the final disposal of the garbage. It is understood that the contractor feeds 2 100 to 2 500 pigs through the season. The garbage, on receipt at the farm, is loaded into wagons and transported to the feeding lots, where it is scattered on the ground.

The budget for this service within the city amounts to about $80 000. This sum provides for a force of 35 to 45 men, and a fleet of 19 collection trucks. Most of the trucks are equipped with flat platforms, and each truck accommodates two steel tanks with covers. The tanks are 10 ft. long, 2 ft. deep, and 4 ft. wide, with trunnions at each end for unloading. At the unloading dock an overhead motor-driven crane is used to lift the tanks and empty the contents into the cars by means of a reversing bridle.

During 1925, the use of horses for garbage collection was discontinued. Furthermore, motor trucks are more desirable than tractor-drawn trailers, on account of their superiority in maneuvering in and out of alleys. Tractor-

* Director of Public Service, Grand Rapids, Mich.

drawn equipment has been found unsafe in the hill districts. For these reasons the Department has standardized more or less on motor-truck equipment.

During 1923 and 1924, an average of about 1 400 tons of garbage per month was collected, the maximum tonnage occurring in September (2 000 tons) and the minimum in February.

The net cost of operating the Department was $73 431 in 1923, and $74 712 in 1924. In each year revenues of about $3 700 were received. The net cost per ton was $4.33 in 1923, and $4.46 in 1924, representing a net cost per capita of $0.49 in 1923, and $0.47 in 1924.

The Superintendent for this Division is paid a salary of $2 100 per year, and the truck drivers and helpers are paid at the rate of $5 per day; after six months' employment at $4.50 per day.

Kenneth Allen,* M. Am. Soc. C. E. (by letter).—In the City of New York, the separation of garbage, rubbish, and ashes and the disposal of refuse in the Boroughs of Manhattan, The Bronx, and Brooklyn, is under the jurisdiction of the Department of Street Cleaning. In the Boroughs of Queens and Richmond refuse disposal is under the Bureau of Street Cleaning of the Borough. Ashes are disposed of separately in Queens, but in Richmond no separation is required.

In 1924, of the total refuse produced, 1 729 213 cu. yd. of garbage and 3 224 486 cu. yd. of ashes were dumped at sea and 3 840 248 cu. yd. of refuse, mostly garbage and rubbish, with 5 941 tons of solid trade wastes, were incinerated. The total refuse includes, in addition, 3 949 545 cu. yd. of ashes and rubbish taken to Riker's Island and other large quantities, including some garbage, disposed of on inland dumps, and of ashes used in grading streets.

The disposal of garbage at sea has been on such a scale that a brief statement of quantities, methods, and costs involved may not be amiss.

Disposal by Dumping at Sea.—The disposal of garbage and ashes by dumping at sea is probably carried on by the City of New York on a larger scale than elsewhere. Owing to the abandonment of a reduction plant on Staten Island in October, 1918, it became necessary to revert to the disposal of garbage at sea as had been the practice prior to 1896, and, again, as an emergency measure due to the burning of the Barren Island reduction plant in 1906. In 1924, 1 138 952 cu. yd. of garbage and 1 051 335 cu. yd. of ashes from Manhattan and The Bronx and 590 261 cu. yd. of garbage from Brooklyn, were disposed of in this manner.

The operation is as follows: Ashes and garbage (except condemned foodstuffs) are collected and brought to twenty-two water-front dumps (fifteen of which are for ashes only) where the material is deposited in scows or "dumpers" holding about 1 800 cu. yd., or 900 tons. Of these there are twenty now in service: Six bottom dumpers, to replace which would cost about $62 500 each, two old side dumpers costing $50 000 each, and twelve new side dumpers, costing $45 000 each. The bottom dumpers are 134 by 34 ft. and the side dumpers 137 by 34 ft., in plan, with sides 14 ft. high. Each dumper contains

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

eight compartments or pockets, which are emptied by gravity on opening the bottom or side, as the case may be.

After loading, a tow of three or four dumpers is taken by a tug on the first of the ebb tide through The Narrows to the dumping point, designated by the Supervisor of the Harbor. Each scow carries a crew of two men, paid together $9 50 per day, and each tow has a Government Inspector who is taken aboard from a patrol-boat at The Narrows. At this point, the condition of the pockets—whether full or otherwise—is inspected. From The Narrows the tow proceeds to the dumping ground, which, for garbage or mixed garbage and ashes, during the summer, is 20 nautical miles east-southeast* (magnetic) of Scotland Light-Vessel, 22 miles east of Long Branch, N. J., 13½ miles south of Short Beach, Long Island, and about 36½ nautical miles, or 42 statute miles, from the Battery. In winter, dumping is permitted at a point 12 nautical miles east-southeast of Scotland Light-Vessel, and ashes may be dumped 4 miles beyond the light-vessel. At a point about 3 nautical miles beyond Scotland Light the tow is inspected by a Federal patrol-boat both on the outbound and inbound trip, the reports of which, compared with those of the inspection at The Narrows, indicate whether surreptitious dumping has taken place in the interval. As an additional check the passage of tows is reported by the Captain of the Light-Vessel. The tow then proceeds to the dumping point designated by the Inspector, stops, dumps the cargo of each dumper, and returns to port. In favorable weather, that is, when not too calm, it takes about 15 min. to empty a loaded dumper; on the other hand, in a quiet sea, some manual assistance is required. During 1924, 494 tows averaging 2.8 dumpers per tow went to the dumping ground, the round trip, probably 90 or 100 miles, taking ordinarily about 22 hours and costing, including Government inspection, $190 per dumper.

As the average load per dumper is $\frac{1\ 728\ 774}{494 \times 2.8} = 1\ 250$ cu. yd., the cost for disposal is $\frac{190}{1\ 250} = 15$ cents per cu. yd.

TABLE 10.—Material Dumped at Sea in 1924.

From:	Ashes.		Garbage.	
	Cubic yards.	Cost.	Cubic yards.	Cost.
Manhattan and The Bronx	1 051 774*	$42 845	1 138 513	$195 405
Brooklyn	.....	.....	590 261	59 300
Total	1 051 774	$42 845	1 728 774	$254 705

* Including 439 cu. yd. of garbage.

From Table 10 the cost of dumping ashes at sea is seen to be $0.041 per cu. yd. and that of garbage, $0.147 per cu. yd. after loading at the water-front dump.

* By a recent order this has been changed to south-east by east, making the dumping point 21 miles east of Long Branch and 16¾ miles south of Short Beach.

This is probably the cheapest method of disposing of the garbage of a seaport like New York, but it is open to objection as a sanitary measure and has always been considered a temporary makeshift.

A considerable part of the material sinks and is no doubt rapidly disposed of by fish and other marine life, but the floating residue, is unsightly and, with winds within a quadrant of southeast, there is always the possibility of its reaching the shore and defiling the beaches. The Metropolitan Sewerage Commission stated that the rate of travel toward the shore "when the tide was favorable and the wind blowing landward at a rate of about 5 miles per hour" was more than ½ mile per hour, and an inspection of the beaches in 1906 showed evidence of garbage more than 50 miles from the dumping point, most of which was "unquestionably" from this source. The dumping point is now farther from shore, but there is complaint of occasional pollution of bathing beaches from drifting garbage the source of which will be the subject of special study during 1926.

The City recognizes the inadvisability of continuing this method of disposal and is extending its policy of incineration so that within a few years the elimination of dumping at sea may be expected. A serious difficulty in most cities, but one acutely encountered in Manhattan, is the strong and effective opposition of local interests to establishing a garbage plant in the neighborhood—a matter that has already caused much delay; but with the abandonment of reduction at the Barren Island and Staten Island plants and the opposition to dumping at sea, the City may be said to be fairly committed to the policy of incineration for the disposal of its garbage.

TABLE 11.—Types of Incinerators Used.

	Number and Nominal Capacity, in Tons per Day.				
	Decarie.	Sterling.	Heenan.	Heenan-Froude.	Total.
For Garbage and Rubbish:					
Manhattan	1 –300	1–320	....	.....	2 – 620
The Bronx	.....	.....	None	.....	.....
Brooklyn	.....	1–100	....	1–100	2 – 200
Queens	1 – 70 4 –100 3*–150	.....	....	.....	8 – 920
For All Refuse:					
Richmond	.....	.....	1–50	1– 50	2 – 100
Total	9*–1220	2–420	1–50	2–150	14*–1840

* One of these under contract.

Disposal by Incineration.—In the entire city there are now twelve incinerators handling garbage and rubbish; sixteen handling rubbish only;* and three handling all refuse. Omitting one of the last, which is experimental in character, those handling garbage are of the types noted in Table 11.

Others are in contemplation in most of the Boroughs, but in Manhattan, especially, great difficulty has been experienced in obtaining sites acceptable

* All in Brooklyn.

to property owners. As new incinerators are brought into operation the quantity of garbage dumped at sea will be reduced until this method of disposal can be entirely abandoned. According to the Hon. Alfred A. Taylor, Commissioner of Street Cleaning, if funds and sites are made available this may be accomplished in about four years.

The incinerators handling rubbish only will take care of 50 cu. yd. per hour, the twelve handling rubbish and garbage have a total nominal capacity of 1 740 tons per day, and the three handling all refuse may be put together at about 150 tons per day. The garbage disposed of in this way in 1924 in Manhattan amounted to 109 297 cu. yd.; in Brooklyn, 56 410 cu. yd.; and in Richmond, 26 437 cu. yd. In the Borough of Queens, 105 633 cu. yd. of straight garbage, 608 949 cu. yd. of rubbish, and 60 610 cu. yd. of garbage and rubbish mixed, were incinerated.

Of the incinerators mentioned, those in Brooklyn are owned and operated under a yearly contract by the Brooklyn Ash Removal Company. Within the past two or three years, two 100-ton incinerators for burning garbage and rubbish have been placed in operation, while sixteen furnaces of about 100 cu. yd. capacity per day for rubbish only, are situated about the Borough. Each of these plants disposes of what remains after picking over 1 000 to 1 500 cu. yd. of rubbish per day.

Each unit is hand-stoked and has three grates of a total area of 72 sq. ft., with corresponding ash-pits. Ashes are used for filling land at Corona, in the Borough of Queens. It has been difficult to obtain such complete separation that the garbage and other putrescible and combustible material left in the ashes will not produce a nuisance in the vicinity of the dump. Mr. R. G. Collins, Jr., Chief Engineer of the Brooklyn Ash Removal Company, has therefore devised a plan which may be generally adopted by his Company and which is in operation at a plant at Sutter Avenue and Junius Street, Brooklyn. This is the experimental plant to which reference has been made.

All refuse except straight steam ashes is first dumped into a pit from the pick-up trucks. From this pit, it is fed to a moving belt from which paper, rags, boxboard, and metal are salvaged and the metal removed by a magnetic separator, the residue being fed from the belt to an Illinois stoker chain grate traveling at a rate of from 30 to 120 ft. per hour. The grate surface is 6 by 30 ft. To obtain uniform incineration the material is kept stirred and evenly distributed by two "agitators" consisting of water-cooled pipes rotating slowly in a direction opposite to that of the grate, as spokes on a transverse horizontal axis. A forced draft applied through tuyères spaced 2 ft. apart under the grate insures complete incineration without auxiliary fuel. The residue is received at the end of the grate and removed in a lateral direction by a drag scraper delivering to a car or truck. As observed, this residue was fine ash without clinker, and contained some cans and small metals that escaped removal. Furnace gases pass through an old furnace improvised as a combustion chamber and thence to a 100-ft. steel stack screened at the

top. This plant takes about 50 cu. yd. per hour, the weight per cubic yard varying from 300 to 400 lb. in summer to 900 to 1 200 lb. in winter.

Advantages claimed for this type of incinerator are: (1) The ability to destroy the putrescible and combustible material received with the poorly separated pick-up collections, thus avoiding a nuisance at and near the dump; and, consequently (2), the necessity for separation; and (3) complete incineration with salvage of valuable material and a minimum of manual labor.

TABLE 12.—Refuse Collected, in Cubic Yards.*

From:	Ashes.	Garbage.	Rubbish.
Manhattan	2 944 359¾	930 246½	2 496 361½
The Bronx	690 259	317 483	519 862
Brooklyn	2 173 151¾	590 261½	2 461 936¾
Total	5 807 770½	1 837 991	5 478 160¼

* The number of tons cannot be stated, but weights per cubic yard that have been stated or assumed by various authorities, although varying somewhat, lead to the conclusion that both ashes and garbage in these boroughs average about 1 100 lb. per cu. yd. and that rubbish, although subject to a greater variation, may be fairly assumed at about 150 lb. per cu. yd. The tonnage for 1924, therefore, would amount to 4 616 030, or 12 630 tons per day, from these boroughs.

For much of the information given, the writer is indebted to Messrs Alfred A. Taylor, Commissioner of Street Cleaning, and E. P. Greene, Superintendent of Final Disposition, City of New York; Carl Gerks, Superintendent of Street Cleaning, Borough of Queens; Theodore S. Oxholm, M. Am. Soc. C. E., Consulting Engineer, Borough of Richmond; and R. G. Collins, Jr., Chief Engineer, Brooklyn Ash Removal Company.

Addendum.—The quantity of refuse handled in 1924 in the Boroughs of Manhattan, The Bronx, and Brooklyn is shown in Table 12.

TABLE 13.—Quantity, in Cubic Yards, and Disposition of Refuse from Manhattan and The Bronx in Each Month, 1924.

Month.	To Sea.		To Riker's Island.	To 56th Street Incinerator.	
	Garbage.	Ashes.	Ashes and rubbish.	Garbage.	Rubbish.
January	80 639	109 124	423 546	.....	392
February	65 306	98 878½	410 802	.....	6 818
March	66 268	109 330½	415 914¼	2 632	31 714
April	80 125	84 550	381 537¾	2 130	38 206
May	104 039	62 352	343 751¾	12 407	39 109
June	117 677	29 570	340 304	13 614	39 056
July	120 952	35 888	375 169¼	13 492	49 600
August	119 434	47 420	294 631¾	13 042	52 020
September	119 630½	65 631½	202 040¼	12 768	50 758
October	120 155	83 470¾	249 099½	12 624	52 728
November	81 488	127 533½	244 842	12 288	29 438
December	63 174	197 636½	267 906¼	11 300	57 546
Total	1 138 951½	1 051 335¼	3 949 544¾	109 297	467 385

In 1924, the refuse from Manhattan and The Bronx was disposed of as follows:

1 138 951½ cu. yd. of garbage was dumped at sea, 19* nautical miles in summer and 12 nautical miles in winter, east-southeast of Scotland Light-Vessel;

1 051 335 cu. yd. of ashes were dumped 12 nautical miles beyond Scotland Light-Vessel; and,

3 949 545 cu. yd. of mixed ashes and rubbish were disposed of by dumping on Riker's Island.

The quantities thus disposed of each month are given in Table 13.

TABLE 14.—Disposition of Refuse from Brooklyn for Each Month, 1924, in Cubic Yards.

Month.	To Sea.	To Land Dumps.	To Incinerator.
	Garbage.	Ashes.	Rubbish.
January	35 697½	250 474	185 024
February	31 944	231 734	157 470
March	37 254½	250 218½	182 000½
April	41 855½	204 344	204 194
May	52 779	161 794½	229 157
June	58 552½	130 779	222 030½
July	67 590	126 763½	238 083½
August	61 529½	111 242½	206 056½
September	64 313	112 068	207 371½
October	55 629	149 779	225 304
November	42 551½	193 413	197 505½
December	40 565	250 541	207 739
Total	590 261	2 173 151	2 461 936

* This was increased to 20 nautical miles in 1925.

TABLE 15.—Quantity of Refuse Collected, in Cubic Yards, and Disposed of, in Borough of Richmond in 1924.

Month.	Collected, in cubic yards.	Household Wastes.			Trade Wastes.	Total.
		To Land Dumps.	Incinerated.		Incinerated.	Incinerated.
		Cubic yards.	Cubic yards.	Tons.	Tons.	Tons.
January	12 841	10 677	2 164	1189.3	565	1 754
February	11 265	9 429	1 836	1007.7	489	1 557
March	12 106	10 262	1 844	1107.8	503	1 611
April	11 235	9 668	1 567	880.0	430	1 310
May	15 033	13 023	2 010	961.7	493	1 455
June	10 289	8 144	2 145	1072.7	391	1 464
July	10 871	8 681	2 190	989.8	450	1 440
August	10 491	7 718	2 773	1059.2	567	1 626
September	10 036	7 060	2 976	1132.4	499	1 631
October	10 792	8 556	2 236	1133.1	327	1 460
November	11 516	9 026	2 490	1091.2	579	1 670
December	14 916	12 710	2 206	1096.1	648	1 744
Year	141 391	114 954	26 437	12 781.0	5941	18 722

In Brooklyn, 590 261 cu. yd., or 324 644 tons, of garbage, including some ashes, and 2 173 151 cu. yd., or 1 195 233 tons, of ashes, were dumped at sea; and 2 461 936 cu. yd., or 135 307 tons, of rubbish were disposed of by incineration. The quantities for each month are given in Table 14.

In the Borough of Queens 1 563 000 cu. yd. of refuse were collected in 1924, of which 780 257 cu. yd. were disposed of on inland dumps and 8 250 cu. yd. in grading streets, while 775 193 cu. yd. were incinerated.

In Richmond, all refuse is destroyed in incinerators or disposed of without separation on land dumps. Of the 141 391 cu. yd. collected in 1924, 26 437 cu. yd., or 12 781 tons, were incinerated and 114 954 cu. yd. disposed of on four land dumps. In addition to the 12 781 tons of household refuse, there were 5 941 tons of trade wastes, making a total of 18 722 tons of refuse handled by incineration—an average of 51 tons per day. Table 15 shows the quantity collected and disposed of in 1924.

The average weight of mixed household refuse per cubic yard varied from 760 lb., in September, to 1 200 lb., in March, with a yearly average of 966 lb.

Harrison P. Eddy,* M. Am. Soc. C. E.—The statement with reference to the general application of a single process to different conditions is very sound. It must be self-evident that the conditions in the different municipalities vary greatly. In his paper Mr. Greeley presented a summarized statement of the general conditions and methods of procedure, which establishes, perhaps for the first time, a basis or starting point for the consideration of garbage disposal that will be of great value to cities confronted by this problem.

One point in particular in his paper is of especial importance—the question of operation. As in many municipal undertakings, it is comparatively easy to secure the capital allowance for construction, but it is exceedingly difficult to obtain the funds for efficient operation. Included in that should be the matter of general upkeep and maintenance.

It is possible, with an inferior plant, to secure relatively good results if there is intelligent and efficient operation. Proper operation is probably the weakest point in the garbage and refuse disposal problem with which municipalities are confronted. The same problem is encountered in the matter of sewage disposal. Plants are built and then the funds for their proper maintenance and operation are rarely forthcoming.

One who is familiar with the plant at Rochester, N. Y., will recognize the soundness of that statement. After the plant had been in operation for relatively a few years (perhaps not more than five), that is, when Mr. Lewis first went there, it was almost on the point of being shut down because it could not be operated. That was largely due to lack of reasonable upkeep, of reasonable care, and of replacement of parts that were not functioning as they should.

Visiting that plant a year later one would hardly recognize it because of the repairs that had been made and the efficiency that had been obtained in

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

operation. That accounts, in large measure, for the high maintenance cost in 1924 (about $40 000 or $60 000 for repairs), whereas during 1925 this had been reduced to about $20 000.

The same is true with every method of disposal. Within two years complaints have been registered against a hog-feeding plant in Massachusetts. The State Department of Health had the matter under investigation. There were indications that the plant would have to be shut down, but by careful operation it has been continued and apparently it is operating successfully, as the complaints have ceased.

Similarly, with incineration, if the plant is not reasonably well operated, unsatisfactory results are certain to be obtained. It is possible that the matter of maintenance and operation is somewhat more difficult in a complicated and intricate plant, such as that described by Mr. Lewis, than in some of the others.

As a rule, incineration plants handle mixed refuse consisting of rubbish and garbage, rather than garbage alone. Misunderstanding of this fact might lead to a little confusion in a consideration of the several papers presented. Undoubtedly the most important feature of garbage disposal is the operation of the equipment provided.

L. L. Tribus,* M. Am. Soc. C. E.—There is a great deal in the Symposium that is worthy of discussion. One point made by Mr. Greeley in particular is a practical one that engineers who have such municipal problems in hand are constantly meeting, namely, as to how the average community selects a garbage destructor.

This is how it is usually done. An enterprising representative of some furnace will get the ear of an official and tell him that in such and such a place there is a very fine plant. The next step is a junketing expedition of inexperienced men who go to see it. The quality of the cigars is fine, the sandwiches, the salad—everything is excellent. The agent who puts up the best exhibit of this sort, wins for his company. After their virtual decision, the officials call in an engineer. If he is in accord with their determination he is just the finest fellow on earth, and the type of plant is adopted; if not, well the veil can be drawn. There is a great deal more of truth than fun in this method and practice: there is no need of enlarging upon it.

The lack of care in the operation of equipment has been mentioned. All engineers have experienced such mismanagement. It makes little difference how well a sewage or garbage disposal plant is built, how "fool-proof", how perfect. After the first send-off and excitement are over, interest is gone. Operators will not give the plant the proper attention; it deteriorates, something happens. If there is intelligent official management, it is put in shape before it gets entirely out of control. If such management is lacking, it goes to pieces.

A great reduction plant was built on Staten Island, New York City—approximately 240 Cobwell tanks, splendidly constructed to handle 1 000 to

* Cons. Engr. (Tribus & Massa), New York, N. Y.

1 500 tons of garbage per day. Why did it not succeed? Although the speaker lives eight miles away, the stench was so great at times that it was necessary to close the windows.

He examined the plant many times, observing receptacles with broken inspecting glasses, so that, when the steam was turned on, the vapors naturally leaked out. When many such leaks existed, it made quite an accumulation of odors, and they could not be kept in the building.

Then, again, scow after scow—from 15 even to 20 in number—lay at the wharf for days with garbage, sometimes even a week old or more on arrival; rotting, filthy stuff, which, although it "smelled to heaven", the plant could not handle promptly.

The tankage building was not big enough to hold the cooked garbage, and, therefore, it was piled on the ground to take the rains and the sun, and give off generously of its caramel odors. That is why the great plant broke down—accumulated mismanagement, and planned with too much optimism as to capacity.

Extensive operating losses were due to the lack of ability to recover the solvents. However, with proper construction, and painstaking management, such a plant might have succeeded, like those at Detroit, Mich., and Rochester, N. Y.

J. F. Jackson,* M. Am. Soc. C. E.—These papers cover the field of garbage disposal from almost every angle, but some minor matters were not brought out which should receive consideration.

In Connecticut there are only two refuse disposal plants, one a reduction plant and one a destruction plant. In the majority of cases garbage is separated from refuse, most of the garbage is fed to the hogs, and the refuse is used for filling. One town uses a sanitary fill. The cost ranges from $0.14 to $1.56 per capita per year which shows the variation in the methods of collection and disposal. In many instances the hog farms are located in a different political subdivision from that in which the garbage is collected, and the work is done under contract. Where it is done by private contractors supervision ceases at the boundary line of the municipality. Few hog farms are conducted on a sanitary basis, and complaints are numerous and frequent. Consequently, the administration duties of the State Department of Health are increased and complicated.

It will be recalled that when control and regulation of the production of milk was started, it was confined at first to testing the milk as it was brought into the cities. Afterward, through the investigation of sources of communicable disease, this supervision was extended to the farms where the milk was produced, controlling the examination of the cows and the sanitary conditions in and around the stables.

Likewise, the supervision of municipal departments having in charge the collection and disposal of garbage could well be extended to include farms on which it is used. It is true that in some cases this is done, but only in a perfunctory manner. At a few municipal farms the arrangements for feeding

* Cons. Engr., New Haven, Conn.

the garbage to hogs are modern, and the sanitary conditions are satisfactory. At most farms lying outside the limits of the municipality they are quite otherwise and, at many, they are deplorable. In investigating slaughter-houses in Connecticut conditions were found that were positively shocking. This is a phase which, although minor, is very important as regards sanitation.

Another analogous feature in the disposal of garbage and the production of milk is the container. It is only in quite recent times that attention has been paid to the milk bottle, its cleaning and capping; but to-day this is quite an important sanitary feature of milk production.

Practically every one at one time or another has revolted at the disgusting condition of the ordinary garbage can. A few favored ones have escaped it by installing house incinerators and testify eloquently to this improvement. Some time must elapse before these will be made a part of the permanent fixtures of a house, but with the trend toward permanent installation of washing machines, refrigeration, etc., the incinerator's place in building construction can be seen in the not distant future. In the meantime much improvement can be made in the control of collection and disposal, and in the design and location of the receptacle.

William A. Bassett,* M. Am. Soc. C. E. (by letter).—Previous to 1922, the Village of Scarsdale, N. Y., like many other residential suburban communities, was dependent on private contractors for the collection and disposal of its municipal wastes. This service at that time was handled by about a dozen individual contractors, some of whom restricted their collection to garbage alone, and the cost to the individual householder ranged from about $3 to $10 per month. Moreover, a considerable part of the population of that village was not furnished with this service so essential in communities changing from rural to urban characteristics.

Recognizing the need for unified service in this matter, under suitable control, a committee of the Village Board working with Mr. Boniface, the Village Engineer, made an extensive study of the problem of waste collection and disposal for residential communities, and as a result of this study, a collection force was organized, suitable equipment purchased, and the construction of a Beccari garbage disposal plant was authorized. There were sound reasons for selecting this type of plant. Its simplicity, low cost of operation and maintenance, adaptability for extension through the construction of additional units, and apparent freedom from nuisance of any kind resulting from its operation seemed to indicate its peculiar suitability for a high-class residential district such as Scarsdale.

Conditions attending the operation of the plant have not fulfilled expectations in so far as freedom from nuisance, caused by objectionable odors, is concerned. As pointed out by Mr. Boniface, after the nitrifying cycle has been completed, and the material in the cells has been made ready for removal, the discharge of the cells is accompanied by highly obnoxious odors. Moreover, the material in the cell has a high moisture content, and is so compacted as to be difficult to handle. Apparently, the obnoxious odors are due

* Prof., Municipal and Industrial Research, Mass. Inst. Tech., Cambridge, Mass.

to the retention of the gases incidental to fermentation by the moisture in the mass. After the material taken from the cell is dried in the open air, it is apparently stable and odorless. Unless these unsatisfactory conditions of operation can be corrected, there is grave doubt as to the applicability of the Beccari system of disposal, at least in its present form, to meet American conditions.

The reasons for this are not hard to find. The quality and characteristics of garbage produced by communities in the United States differ materially from those found in Italian cities, in which the Beccari system is at present in successful operation. American garbage is high in grease, animal fats, and moisture; that produced in Italian cities is decidedly low in these qualities, particularly in its moisture content. This is due, in part, to the salvaging methods used in Italy in handling garbage before it is introduced into the cell. Moreover, it is stated that at the Italian plants the material when removed from the cell is dry, dark in color, and of about the consistency of humus or leaf-mould. It would not appear probable that the richness in fats and greases of the garbage produced by American cities would have any unfavorable influence in the operation of the nitrifying process. In fact, it is believed that the contrary might prove true. However, there appears to be little question but that the high moisture content of the Scarsdale product re-acts unfavorably on the process, and has been the main contributing factor in producing the nuisance conditions.

It is quite probable that some modifications in the present design of the Beccari plant at Scarsdale should be made in the construction of other plants, but it is believed that the main feature required for successful operation of this process is to effect a substantial reduction in the moisture content of the garbage before the latter is placed in the cell.

Samuel A. Greeley,* M. Am. Soc. C. E. (by letter).—The trend of the discussions emphasizes again the difficulties in properly relating the engineering and administrative aspects. Mr. Eddy justly comments on the importance of operation and on the fact that sufficient funds for satisfactory operation are not always available. It is true, as he states, that careful operation is necessary to avoid unsatisfactory results with any method of disposal.

Mr. Tribus' comments bring clearly to mind the large economic loss involved in the abandonment of the Staten Island reduction plant; which, in some way, should have been avoided by a sound correlation of engineering and administration.

Mr. Jackson brings out the tendency in many hog-feeding projects of shifting the disagreeable features of disposal to a different political subdivision. Whereas this usually isolates the hazards of offensive conditions, there is little justification for burdening outlying districts with a nuisance from the garbage collected in other political subdivisions without compensation. As he states, the supervision of the municipal departments could well be extended to apply to the hog farms.

* (Pearse, Greeley & Hansen), Chicago, Ill.

Mr. Bassett's remarks illustrate the need of properly adjusting the method of disposal to its environment. The fact that a particular method is satisfactory under one set of atmospheric and collection conditions, does not mean that it will be satisfactory under different conditions.

Mr. Sperry, in commenting on the disposal for Grand Rapids, Mich., does not show any provision for sanitary control of the hog farm, thus throwing the burden on the outside territory.

Mr. Allen's data picture the enormous quantities involved. They further illustrate that the cheapest disposal is not always the one to be adopted and that more cost is justified in order to gain a more sanitary disposal. The neighborhood's objection, which he mentions, of having garbage plants located in its midst, is more especially true of large plants. A careful engineering survey should be made to determine the best sites so as to justify the locations selected. Mr. Allen also states the difficulties encountered in proper separation, giving rise to nuisance on ash dumps of decomposing garbage. This illustrates the need for sustained effort to instruct householders in the proper treatment of garbage, ashes, and other refuse.

Edward D. Rich,* M. Am. Soc. C. E. (by letter).—Since the writer's paper was presented several important changes have taken place in the disposal of garbage at Lansing, Mich.

Effective May 1, 1926, the charge for can rental was increased from $1.00 per year to $1.50 per year. This increase was not justified in the light of the receipts and expenditures shown in Table 16.

TABLE 16.—Lansing, Michigan, Can Fund for Six Years, 1921-1926, Inclusive.

Year.	Receipts.	Expenses.	Profit.	Loss.
1921	$8 403.95	$7 528.32	$875.63	.....
1922	11 058.90	10 529.75	529.15	.....
1923	11 675.14	11 587.31	87.83	.....
1924	12 884.25	9 203.60	3 680.65	.....
1925	12 412.25	14 408.50	.....	$1 996.25
1926	17 625.48	9 857.93	7 767.55	.....
Total	$74 059.97	$63 115.41	$10 944.56	.....

It will be noticed that there was a profit in the fund every year except 1925. An increase in expense of about $4 000 in that year was due to an extra large supply of surplus cans purchased during that time. Of the accumulated surplus of nearly $11 000 in this fund, about $3 000 has been used for repairs at the farm and the balance remains in the fund.

Including data covering the years subsequent to those given in Table 3, there results Table 17, showing the costs of collection through 1926.

Table 17 places the writer in the embarrassing position of being obliged to contradict his previous figures. This is due to the lack of specific accounting

* Director, Bureau of Eng., Michigan Dept. of Health, Lansing, Mich.

which makes it difficult to determine the real standing of the garbage collection and disposal business of the city. The figures given in Table 17 are as reliable as can be obtained.

TABLE 17.—Cost of Garbage Collection, Lansing, Michigan, 1921-1926.

Year.	Cost.	Population.	Patrons served.	Population served.	Percentage of population served.	Cost.		
						Per capita total population.	Per capita population served.	Per patron.
1920	$36 245.54	58 632	.....	.....	....	....	....	....
1921	35 869.03	61 242	7 600*	31 550	51.5	0.59	1.14	4.67
1922	35 951.47	63 852	9 210	38 200	59.8	0.56	0.94	3.90
1923	43 038.31	66 462	10 911	45 300	68.2	0.65	0.95	3.95
1924	44 839.76	69 072	10 811	44 850	64.9	0.65	1.00	4.15
1925	56 482.78	71 682	11 853	49 200	68.6	0.78	1.15	4.77
1926	56 690.78	74 292	11 503	47 750	64.3	0.76	1.19	4.93

* Estimated.

Disposal.—In 1925 an additional farm of 95 acres was purchased at a cost of $13 300. This was needed for the disposal of refuse from the feeding floors, but its purchase was made more particularly because of the bitter complaints against the operation of the plant raised by the owner of the property. In the spring of 1926 the State Supreme Court granted a stay of proceedings with the understanding that the City of Lansing should have the opportunity of improving operations during the summer months so as to eliminate nuisances if possible to the satisfaction of the neighbors. After this a hearing would be granted in the Circuit Court of Eaton County and a report of this hearing would be submitted to the Supreme Court to aid it in its final decision.

The improvements made by the City at the pig farm were chiefly in the method of burial of the refuse. This was accomplished by the use of a gasoline steam shovel instead of the road machine and hand labor employed before. Few if any changes were made in the actual feeding operations because these had been conducted as carefully as possible previously.

Several new buildings were constructed with the idea that the Supreme Court decision was very likely to be favorable to the City. These buildings included a hospital for the treatment of sick hogs, 40 by 80 ft., and a quarantine house, 40 by 65 ft.; also a pump house, 40 by 12 ft., which contains a well, pump, and pressure tank. The old quarantine house was not large enough and was not favorably located.

During July and August, 1926, the writer made an inspection for the City on each alternate day to observe what odors, if any, were discernible on the public roads surrounding the piggery. Records were made of the temperature, state of the weather, and direction and velocity of the wind. During the two months on only one occasion was there a distinct smell due to the piggery. On several other occasions slight traces of pig odor were noticeable, but not marked enough to justify any one in complaining of the nuisance. On the

other alternate days a member of the Faculty connected with the Animal Husbandry Department of the Michigan State College made the inspections. Both reports were similar.

In spite of the results disclosed by these observations, several witnesses at the hearing swore that the nuisances were worse than ever before. This was manifestly untrue. Finally, the Supreme Court was obliged to decide against the City. The decision of the Lower Court was affirmed and the injunction granted, to become effective May 1, 1927, and the City was ordered to discontinue the use of this property for pig-feeding purposes.

The City sought other sites on which to continue feeding; but, being unsuccessful in securing a site acceptable to all members of the Council and being in receipt of numerous complaints against the establishment of a piggery by neighbors of the suggested sites, accepted a proposal from a private company to take all the city garbage and dispose of it by feeding. It seems unfortunate indeed that such a well-operated disposal plant should be put out of business as a result of unfortunate circumstances in dealing with the neighbors of this pig farm. Apparently, certain methods used by the City were more responsible for the belligerent attitude of these farmers than the odor itself.

As near as can be estimated the cost of burial of the refuse from the feeding floors by the partial hand method described in the paper was about $1.50 per cu. yd. and by the gasoline shovel method, about $0.60 per cu. yd.

The writer was anxious to obtain a complete record of the operation of the pig farm covering the six years of its existence, but found that the City Comptroller's books were kept in a way that would have entailed a great amount of labor in separating and classifying the items. However, a statement showing the total receipts and expenses for all purposes covering the six years is given in Table 18, with the profit and loss corresponding to each year.

TABLE 18.—Cost of Garbage Disposal, Lansing, Michigan, 1921-1926.

Year.	Receipts.	Disbursements.	Profit.	Loss.
1921	$45 503.46	$58 130.39	.....	$12 626.93
1922	46 352.45	48 420.97	.....	2 068.52
1923	54 012.01	54 303.65	.....	291.64
1924	70 676.41	66 638.45	$4 037.96	.....
1925	84 670.15	81 964.84	2 705.31	.....
1926	116 278.55	117 981.91	.....	1 703.36
Total	$417 493.03	$427 440.21	$6 743.27	$16 690.45

Table 18 indicates a total loss for the six years of $9 947.18. These expenses, however, include many improvements, buildings, equipment, legal expenses, and purchase of land. The legal expenses for the lawsuit were more than $4 000. Somewhat more than $12 000 was spent during 1926 for improvements on land and buildings, including a new water supply and the structures already mentioned. The cost of the gasoline shovel was $5 225. In the event of the sale of the farm very little return can be realized on

some of the improvements which cost the City a considerable sum. The gasoline shovel is available for use on other work and so does not suffer any appreciable depreciation. The buildings have small salvage value, but perhaps something can be realized on the water supply equipment. Some return on fencing and other improvements may appear in the increased value of the land when it is sold, but probably not enough to approach their cost.

As a result of six years' operation of the piggery, the City finds itself in possession of 215 acres of land estimated by a reliable real estate man to be worth $225 per acre. This amounts to $48 375. In addition, there is equipment estimated at $6 500 and a flock of sheep at $900, all of which are salable, making a total of $55 775. There is an outstanding mortgage of $10 000 on the land, leaving $45 775. Deducting from this the deficit of approximately $9 950, the profit for the six years is $35 825, or nearly $6 000 per year. This profit was actually realized. If the City had been allowed to continue its pig farm the showing would have been much more favorable for the reason that the expenses of the lawsuit and heavy depreciation on the buildings due to the discontinuance of the business would have been avoided. This difference would doubtless have amounted to an additional $12 000 or $15 000 profit.

It appears that two major lessons can be drawn from the experience of the City of Lansing in its operation of the pig farm: First, a farm should be located in a more isolated spot, particularly avoiding such a high-grade farming community; and, second, great care should be taken in all cases of this kind to treat the neighbors in a considerate manner. If this is done, their friendship can be obtained rather than their enmity. By furnishing them a liberal amount of fertilizer free, or even at some slight expense, real or fancied objections can be forestalled and the city saved not only a vast amount of anxiety, but also a serious financial loss.

John V. Lewis,* Esq. (by letter).—The writer's paper has tended to create an interest in the Cobwell System and the results to be obtained from it, rather than a wide discussion of the subject itself. The fact that the system is a comparatively new one; that it has not been used extensively by municipalities to date; and that only three such plants are now in actual operation, probably accounts for the limited comments.

In their discussions, Messrs. Eddy and Tribus have mentioned the major factor which spells success or failure for the Cobwell System from a sanitary standpoint, namely, proper maintenance and upkeep. While reasonable care and attention are essential for the satisfactory operation of any type of refuse disposal plant, the complicated and intricate system requires more constant observation and upkeep. This matter also has a direct bearing on the question of economy or net cost of operation. To an equal extent, however, the market value of the by-products will decide the question of what type of disposal plant a municipality should install.

* San. Engr., in Chg. of Refuse Disposal Div., Dept. of Public Works, Rochester, N. Y.

Unfortunately, the market for the by-products of reduction plants has been very unstable during the past few years, tending to bring the mixed refuse destructor type of plant into greater use in the United States. The net cost of high-temperature incineration as outlined, for instance, by Mr. J. A. Burnett, in his paper* on "High-Temperature Incineration at Toronto, Ontario, Canada", must prove more interesting to the average public works official than the same operating cost for a reduction plant with the tankage fertilizer and grease market at its present level.

* See p. 824.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1648

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION*

By Roy G. Finch,† M. Am. Soc. C. E.

With Discussion by Messrs. Frederic H. Fay, Maurice W. Williams, T. Kennard Thomson, Harry Taylor, E. P. Goodrich, F. Lavis, J. K. Finch, E. E. King, Frank L. Bolton, and Roy G. Finch.

During 1925 the New York State Barge Canal and its operation was discussed at considerable length throughout the State of New York. This was due in part at least to the functioning of a special legislative commission created for the purpose of ascertaining why the canal was not handling a greater tonnage and of determining what could be done to stimulate traffic. The effectiveness and economic feasibility of this type of waterway have been questioned, although prior to its construction in 1903 it was pronounced by engineering experts of recognized ability as the proper type of undertaking to afford a superior waterway connection between the Great Lakes and the seaboard.

History of New York State Canals

In order to visualize properly present-day conditions it may be helpful to retrace briefly the history of the canals in New York State. The first waterway improvements were made by a private company chartered in 1792. By 1798 the natural streams of the State had been improved to facilitate traffic to a considerable extent, but it was not until 1817 that the State actually undertook the construction of the Erie Canal which was opened in 1825. The channel was 4 ft. deep and 28 ft. wide and floated boats carrying 30 tons.

* Presented at the meeting of the Waterways Division, New York, N. Y., January 21, 1926.

† Cons. Engr., Albany, N. Y.

After the building of this canal the City of New York grew at an extraordinary pace and soon displaced Philadelphia as the Nation's chief seaport. So marked was the success of the Erie Canal that a veritable frenzy of canal building spread over the whole country, manifesting itself in New York State by the building of several lateral canals, six within the first decade after the Erie was completed, and four more within the next four years. To meet the constantly growing demands of traffic, the Erie and its main branches were enlarged from time to time. In 1862 the Erie Canal had a depth of 7 ft. and was capable of floating boats carrying 240 tons, a large increase compared with the first boats of 30-ton capacity on the original canals. By 1883, the year in which it was created a free canal by the abolition of tolls, the Erie Canal had earned a net surplus of nearly $43 000 000 in excess of its original cost plus the expenses of enlargements, maintenance, and operation.

In 1903, nearly ninety years after the starting of construction on the original Erie Canal, the people of New York State decided again to enlarge the canals by the building of what has been generally known as the "Barge Canal." This consists of the Erie Canal and the three principal branches of the canal system—the Champlain, the Oswego, and the Cayuga and Seneca Canals. The Erie is about 340 miles long and reaches across the State from Troy, on the Hudson River, to Tonawanda and Buffalo, on the Niagara River. The Champlain Canal runs north near the easterly boundary of the State from Troy to Whitehall, the southern end of Lake Champlain, a distance of 63 miles; the Oswego extends from a point on the Erie Canal near Syracuse to Oswego on Lake Ontario, a distance of 24 miles; and the Cayuga and Seneca Canal leaves the Erie west of Syracuse and runs southward, connecting with Cayuga and Seneca Lakes, a distance of 27 miles. Including the Hudson River and the lakes connected with the canals at various points and actually forming a part of the system, the total length of the New York State Barge Canal System is about 800 miles, of which about 70% is either in the beds of natural streams or in lakes.

Physical Features of Barge Canal

From tide-water level at Troy, the Erie Canal rises through a series of locks in the Mohawk Valley to Elevation 420 at the summit level at Rome and descends to Elevation 363 at the junction with the Oswego Canal and then rises to Elevation 566 at the Niagara River, a total ascent of 623 ft. and a descent of 57 ft. The Champlain Canal ascends from tide-water at Troy to Elevation 140 at the summit level at Fort Edward and then descends to Elevation 97 at the entrance to Lake Champlain, a total ascent of 140 ft. and a descent of 43 ft. The Oswego Canal descends 119 ft. from its junction to Lake Ontario. The Cayuga and Seneca Canal has a total ascent of 71 ft.

The channel of the waterway has a uniform bottom width of 75 ft. in earth sections of the land line, 94 ft. in rock, and 200 ft. or more in the beds of rivers and lakes, and has been excavated to a depth of 12 ft.

The locks are of the miter-gate type and have a usable width of 44½ ft. and a length of 300 ft. with a minimum depth of 12 ft. over the miter-sills.

There are thirty-four locks on the Erie, eleven on the Champlain, seven on the Oswego, and four on the Cayuga and Seneca Canals.

The water supply for the western half of the Erie Canal is obtained from the Niagara River and for the eastern half an adequate supply is assured through the operation of two storage reservoirs which feed into the summit level. These reservoirs have a total capacity of more than 6 000 000 000 cu. ft. and are located at Delta on the head-waters of the Mohawk River and at Hinckley on West Canada Creek, one of the branches of the Mohawk. The Champlain Canal is dependent on the waters of the Hudson River for its supply, and the Oswego and Cayuga and Seneca Canals on the rivers that were canalized to form these canals and are fed by the lakes in the central part of New York State. Dams of both the fixed and movable type have been built as a means of controlling the canalized rivers.

The Barge Canal is crossed by about 306 railroad and highway bridges, the greater number of which are of the fixed type having a minimum clearance of 15½ ft. Terminals have been constructed at every city and nearly every village along the line of the canal. The facilities at the several sites vary, but in general consist of docks, wharves, harbors, freight-sheds, mechanical devices, and, in some cases, railroad connections for the interchange of freight between railroad and water carriers. At Gowanus Bay, Brooklyn, and at Oswego, on Lake Ontario, modern grain elevators have been constructed with capacities of 2 000 000 and 1 000 000 bushels, respectively.

To December 1, 1925, the State of New York had spent in the construction of the Barge Canal System and its terminals the sum of $174 258 558, which does not include interest on bonded indebtedness or maintenance cost, which now averages about $7 000 000 per year. So much for the physical structure itself.

Economic Features

The importance of the territory adjoining the Barge Canal is not generally appreciated. More than 70% of the people of New York State live within 2 miles of the waterway, which means that 7% of the population of the United States is within a 30-min. walk of the New York State Waterway System. It is easy to understand what it means not only to the State of New York but to the country at large that the products of these 8 000 000 people and the supplies they need shall have available a cheap means of transportation.

The Canal System as it exists at present has an annual carrying capacity in excess of 20 000 000 tons. During the navigation season of 1925, it carried 2 333 000 tons, which was 15% in excess of the tonnage carried in the highest previous year and a gain of more than 100% over 1918, the first year the Barge Canal was placed in operation throughout its entire length.

This question naturally suggests itself, why is the New York State Barge Canal System carrying only 10% of the tonnage it is capable of moving? Almost every one seems to feel qualified to answer this question but rarely do two people arrive at the same conclusion. In the attempt to diagnose this case one can proceed without fear of contradiction on the accepted basis that water transportation is inherently cheaper than any other form and that the terri-

tory served by this waterway system has a potential tonnage suitable for shipment by canal far in excess of the canal's present capacity.

Development of Traffic

Irrespective of the good physical condition of the Barge Canal or of its proved serviceability, carriers must be present to transport the tonnage if the full benefits of water-borne transportation are to be realized. There is a lack of boats on the New York State Canal System; of this there can be no question. The situation is not unlike that of a railway system which, after having provided its right of way, constructed the roadbed, laid its rails, built its terminals and freight houses, only awaited the arrival of the rolling stock to make it a going concern. Back in the Seventies when the 7-ft. canals were handling 6 000 000 tons of freight annually, there were on the canals between 5 000 and 6 000 boats of 240 tons capacity, drawn by horses and mules. To-day, it is doubtful if there are 800 boats capable of service and in condition to meet the requirements of the underwriters. Why this great decrease in the number of boats?

The agitation for a larger canal started in the Nineties, but it was not until 1903 that the dimensions of this larger canal were decided. During these years the boat owner was aware that sooner or later a canal would be constructed of a size on which the 240-ton boat then in operation would not be the most efficient unit, and naturally he spent nothing on new equipment and the minimum on upkeep and repairs. The canal was fifteen years in the building and during that period the number of boats maintained in the canal service became less and less each year. The boat owners and operators were still undecided as to the proper type of boat that could be used most efficiently on the new waterway—and even to-day there is a wide difference of opinion—which indecision resulted in a still further loss of boats in the service and the abandonment of those which had outlived their period of usefulness or required any extensive repairs. No new boats were constructed to take their place. The World War came on with a great increase in the prices of labor and materials. Priorities could not be obtained for materials for building canal-boats. The Government took over the operation of the canals and for a time no assurance could be obtained that privately constructed boats intended for service on the New York Barge Canal would not be commandeered by the Government and placed on some other waterway. Conditions were such immediately following the close of the war that boat-building operations were not stimulated. Then came a period of great agitation for a still larger canal—a ship canal—following the St. Lawrence route or the Oswego-Mohawk-Hudson route through New York State. Those operating on the present canal or those contemplating such an operation found themselves somewhat in the same position as their predecessors prior to 1903. Should a larger and different type of canal be constructed the boats best adapted for use on the present canal probably would not be the most efficient for use on the new waterway and, therefore, operators rather felt that they should curtail building operations and await developments. Finally, the creation of a State Commission to make a survey and study of the Canal System with a view of recommending what

should be done to stimulate traffic, gave rise to the thought in some quarters that New York State might abandon its canals. The net result of all this has been that capital seems reluctant to finance boat-building operations. In the writer's judgment this lack of carriers is the main reason why the Barge Canal is not transporting the tonnage for which it was designed and to-day is capable of carrying.

Types of Carriers Used

An outstanding example of the use that can be made of the Barge Canal is furnished by the Standard Oil Company. This Company is operating on the Canal System nine steel tank barges the largest of which are 240 by 40 ft. and, on a 9-ft. loaded draft, carry 14 000 barrels of gasoline. Tanks have been constructed at many of the cities and villages along the line of the canal, and the oil or gasoline is pumped directly from the barges to these tanks and distributed by motor truck to the adjacent territory. This Company is constantly extending the scope of its operations; in 1925, tank barges were operated as far west as Buffalo, south to Ithaca on Cayuga Lake, north to Ogdensburg on the St. Lawrence River, and to Rouses Point at the northern end of Lake Champlain. The Standard Oil Company has built its barges to fit the canal and the fact that each year additional barges are put in service and land-station facilities are extended furnishes the answer as to whether the use of the Barge Canal is looked on with favor by this Company.

Other types of craft operating on the Barge Canal include the two steel motor ships of the Minnesota Atlantic Transit Company. These have a length of 258 ft., a width of 42 ft., and operate on a 10-ft. loaded draft. They are classified for Great Lakes and coastwise service and during 1925 operated between New York and Detroit, Mich. Loaded to 10 ft., they have a carrying capacity of 1 200 tons and make the trip between New York and Oswego, on Lake Ontario, a distance of 338 miles, in 72 hours.

The Interwaterways Line operates five steel motor ships, 256 ft. long and of 36 ft. beam, which, on a 9-ft. loaded draft, carry about 1 500 tons. These ships do not operate on the Great Lakes, nor in coastwise service. The running time of these boats between New York and Buffalo, a distance of 507 miles, is between 5 and 6½ days. A round trip from New York to Buffalo, including the discharge and reloading of cargo at Buffalo, has been made in 11 days and 15 min. The cost of these boats is about $175 000 per unit.

The Trans-Marine Line operates its boats in fleets, the equipment consisting of thirty steel barges, five wooden barges, and five Diesel engine tugs used for towing the barges. The barges have a length of 100 ft. and a beam of 20½ ft. Five barges and one tug comprise a fleet which, on a 9½-ft. loaded draft, has a carrying capacity of about 2 000 tons. The cost of such a fleet is about $150 000.

The Munson Line has recently absorbed the New York Canal and Great Lakes Corporation, operating seventy steel barges built by the Government during the war. These barges are 150 ft. long and of 20 ft. beam. Nineteen of these boats are power boats with only part cargo-carrying capacity and they

tow the other barges. Four boats fill a lock and on a loaded draft of 10 ft. have a carrying capacity of about 1 950 tons.

The Brooklyn and Buffalo Navigation Company operates eighteen wooden barges and three cargo-carrying steamers. The barges are 100 ft. long and of 22 ft. beam, and a fleet consisting of five barges and the steamer, utilizing the full capacity of the lock, on a draft of 10 ft. 4 in., has a carrying capacity of 3 600 tons. The cost of such a fleet is about $100 000.

The statements made relative to the different types of boats are for the purpose of identifying the principal types of craft now operating on the Canal System and do not cover all the operating companies or the individual or independent operators who are using the canal.

Physical Conditions

Much misinformation has been spread broadcast relative to the physical condition of the Barge Canal, particularly as to the channel depth. The project depth was 12 ft., and the canal was excavated to that depth. Engineers can readily appreciate the impossibility of maintaining a 12-ft. channel for full project width at all times on a canal which at the outset was dredged to only 12 ft., and which for the greater part of its length is located in the beds of natural watercourses. The original designers had in mind a canal to float boats on a 10-ft. loaded draft and fixed the depth to which the channel should be excavated at 12 ft. Unfortunately, the impression prevails that if boats in a 12-ft. channel cannot be loaded to a draft of approximately 12 ft., there must be something wrong with the canal. Observers state that there is insufficient depth, but cannot give its exact location. Testimony of operators has been given before the Barge Canal Survey Commission showing that in 1925 boats drawing 10½ ft. of water navigated the canal from Buffalo to Troy without difficulty. Even in the face of such proof as to the absence of obstacles in the channel one hears on all sides that if the canal were dredged to its statutory depth boats in greater number would navigate it. During the latter part of the navigation season of 1925 the public press carried statements to the effect that the depth of the canal was only 8 ft. in certain sections. This was not so. At the time the writer knew of no place in the channel between Buffalo and Troy at which a boat drawing 10 ft. could not safely navigate, and such was the contemplated loaded draft for vessels when the decision was made to construct a 12-ft. canal. During 1924 and 1925 the policy was adopted, in removing material under the head of "Maintenance", to excavate 2 ft. below the established grade line, thus affording a channel depth of 14 ft. There is nothing more detrimental to the progress of the canal than erroneous statements constantly being made as to the depth of the channel.

Rail Connections

The establishment of rail connections and pro-rating agreements with rail lines at the canal terminals are of the highest importance. The necessity of this for successful operation of the canal is obvious. All the railroads are not yet willing to look on canals as a medium of transportation, having a place and performing a useful function in the present-day transportation system, and

are not disposed to consider the canals as supplementing the other means of transportation. They are thus depriving the people of the benefits that would accrue from a co-ordination of rail, water, and truck movements. An undesirable condition was created by the New York Central Railroad Company which secured a Court order, preventing the enforcement of the order of the Interstate Commerce Commission for a connection and interchange between canal and rail at the Erie Basin Terminal, in Buffalo. This matter deserves prompt treatment.

Packet Service Possibilities

The consensus of opinion seems to be that the Barge Canal is adapted for packet service, that sufficient tonnage would be offered, and that such service properly conducted would be profitable. Packet lines, however, have not been organized and financial interests apparently are not anxious to support such enterprises. That the State itself should try the experiment of a packet line has been advocated. Unquestionably packet service would offer a means whereby thousands of people, residents of New York State, who contributed to the building of the canal system, might participate directly in the savings from water transportation. The State might not be going too far afield in trying the experiment, and by establishing a dependable packet service over a certain section of the canal and limiting the service to a definite period of time, might demonstrate whether the canal is adapted for such service.

Means for Increasing Usefulness

Placing the canals under a permanent commission has been strongly advocated and the sound argument advanced that a continuing policy for canal management under the direction of qualified engineers must be formulated if the canal is to succeed. The directing head should not change with every new administration. The State must "sell" the canal to the shipping public and by personal solicitation obtain business through personal contact the same as the railroad corporations.

There has never been a time when there was a more crying need for clear thinking in relation to waterways than at present. The writer is unalterably opposed to a penurious policy in the expenditure of money, if any additional expenditure for the New York State Canal System is necessary, but he is just as much opposed to the idea that regardless of what may or may not be accomplished any expenditure for waterways is by any and all means money well spent. Those who believe that waterways transportation is of a distinct value to the country should insist that waterways be not further burdened with questionable expenditures. The most careful investigation should prove beyond a shadow of doubt that the promised benefits are commensurate with the amount to be expended.

Regrettable as it may be, the Barge Canal is accepted as a failure by too many people and as something in which the people of New York State have invested millions of dollars from which they are not receiving any ample or proper return. Not a sufficient number of people participated directly in the savings in freight from the handling of 2 333 000 tons in 1925, and although

it is estimated that the existence of the canal system resulted in a depression of rail rates in New York State which saved railroad shippers $50 000 000 during the year, this indirect saving is not understood and the wonderful asset which the people of the State have in the canal system is not appreciated.

Although the writer realizes that on every occasion the people of New York State have generously voted money for canal improvements, it is nevertheless a fact that many are disappointed in the tonnage transported on the Barge Canal. The unsupportable presumption that canals in New York State are a thing of the past is becoming too pronounced. Before it is determined to ask the people to contribute toward the building of a different type of canal, however, every effort should be exhausted to make the present canal serve its intended purpose. The Barge Canal is not admitted by the people of the Middle West as an outlet to the sea, and unless it can be shown in the not too distant future that a canal of this barge type can be made an adequate outlet to the Atlantic there will be constructed, over some route, a ship canal from the Great Lakes to tide-water and the people of New York State will be called on to pay substantially 30% of whatever is spent on such construction by the Federal Government. Therefore, they deserve to have every agency used to make their existing canals a success.

Future Enlargement

In connection with studies being made relative to a proposed ship canal following either the St. Lawrence River or the New York State route, the writer feels that consideration should be given to the possible benefits to be derived from deepening the present canal from 12 ft. to 15 ft. and increasing the width of channel to a minimum of 110 ft. This would allow boats which make full use of all available space in the locks to load to a depth of 11 ft. 9 in. and not be subject to interference or delays in navigating the canal. Self-propelled barges of more than 2 000 tons capacity could then efficiently operate on the canal and their carrying capacity would be increased from 20 to 30% with practically no increase in operating expense. Such a result would prove a greater inducement to capital to place boats on the canal; and this must be offered, else sooner or later the canal will be abandoned, for a canal to be successful cannot justify itself without boats.

The people of New York State, in view of the millions of dollars willingly voted for waterways, should have definite knowledge and absolute proof that some other type of canal will completely answer the demand before they are again asked to give the vast sum that will be New York's share of the enormous cost of a ship canal. Who can say that the deepening and widening of the present canal may not go a long way in affording the outlet to the sea which the people of the Middle West are now demanding.

DISCUSSION

FREDERIC H. FAY,* M. AM. SOC. C. E. (by letter).—The author has ably described the present condition of the New York State Barge Canal and the apparent reasons why at this time it is used to such a small part of its capacity. Further, he has outlined improvements both in canal structure and in canal operation which should be carried out to make the Barge Canal of the greatest possible usefulness. Because of unfortunate conditions in the past, the canal is even now receiving much undeserved and unfavorable criticism.

In the past, even from its beginning in 1825, the Erie Canal has been of inestimable value in building up New York State and the great Middle West. It was a leading factor in making New York City the first port of the Nation and also in helping it to retain this position.

During the latter part of the Nineteenth Century the railroads were expanding at a rapid rate and reaching out for traffic in every direction. Gradually the greater part of the commerce of the canal was absorbed by the railroads which have retained their grasp on this commerce to the present day. The time has now arrived when the economic needs of the country demand that greater use be made of water routes to the relief of existing rail lines. This is best appreciated by a comparison of the present-day railroad traffic with that of a generation ago. There has been a remarkable increase in traffic on the railroads during the last 30 or 40 years with no comparable increase in railroad mileage. In fact, the last few years have witnessed an actual decrease in miles of road operated. In 1890 the freight ton-mileage on all the railroads in the United States was 76 000 000 000; in 1920, this freight movement was 414 000 000 000 ton-miles, an increase of 443% in 30 years. During this period, the total miles of road had increased only 66% (from 156 404 to 259 941) and the total length of all main-line tracks had increased only 79% (from 166 164 to 296 835 miles). Freight movement, therefore, increased in this 30-year period nearly seven times as fast as the increase in railroad mileage, and five and one-half times as fast as the increase in main-line track capacity.

There is a limit to the freight which can be handled economically by the present railroads even if terminal facilities are enlarged. The time is not far distant when no readily conceivable expansion of the present railroad system will suffice to handle the traffic, and greater use must be made of the economical water routes over which many classes of freight may be transported at costs which are only a fraction of those incurred for movement by rail.

The people of New York State, realizing that its canal system had not kept pace with the growing needs of the country, authorized in 1903 the construction of the new canal, now known as the Barge Canal. Some years later they authorized also the building of public canal terminals at important cities and towns along the route of the canal.

The Barge Canal was completed in 1918 when the United States was at war and conditions throughout the whole country were abnormal. The Fed-

* Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

eral Government took over the operation of the canal and did not relinquish it until 1921. During the period of its construction there was no incentive for the old operators on the Erie Canal to provide modern equipment; they did not know whether the canal would be efficiently managed or that the full depth required would be maintained. On no other canal in this country were operating conditions the same and only by the fullest experiments could the type of boat that would be economical and efficient on the new waterway be determined. It is a long step from the old horse-drawn barge of 300 tons capacity to the modern Diesel electric unit of 1 500 tons capacity. It takes time not only to determine upon and construct this modern equipment, but also to train a personnel for its operation.

All this is being done on the present Barge Canal without the assistance of either the Federal Government or the State. An instance of the time required and the trials necessary for the development of a new waterway route are well illustrated by the barge-line operation of the Mississippi-Warrior Service, under the control of the Federal Government. This Service, after a number of years of Federal operation, is only now "getting on its feet". It cannot be expected that the New York Barge Canal operators, unaided by Government support, can develop the commerce of that canal any faster than the Federal Government can show results from its own experiments.

There are two reasons why a greatly increased use of the Barge Canal is to be expected in the near future. First, the general economic proposition that waterways are now needed for the relief of the rapidly increasing traffic burden on railroads; and, second, the situation that will result from the opening of the new Welland Ship Canal between Lake Erie and Lake Ontario.

The experience in these few early years of Barge Canal operation has demonstrated that types of canal carriers can be provided to transport many classes of freight, particularly bulk commodities, at a cost substantially below that of transportation by rail. Furthermore, if the canal is deepened to 14 ft. throughout its reaches and its canalized river and lake sections, so as to permit the passage of boats built to the full dimensions and full draft permitted by the present locks, the economies of canal transportation will be further increased. With definite assurance that the State of New York will at all times maintain the waterway at its full usefulness, canal operators will be justified in providing the modern fleets necessary for economical operation, and the canal will then unquestionably carry a greatly increased traffic, some of which is carried at present by rail.

With the opening of the new Welland Ship Canal from Lake Erie to Lake Ontario, the Barge Canal route from Oswego, N. Y., to the Hudson River will become a far more important link in the National transportation system than is now generally realized. At present, there are transported annually on the four Upper Great Lakes, in modern large lake steamers, from 100 000 000 to 125 000 000 tons of freight at a transportation cost of one-seventh to one-tenth the cost per ton-mile of the transportation of similar commodities by rail lines. The Great Lakes fleet of vessels, with a normal loaded draft of 20 ft. or more, cannot go east of Lake Erie owing to the limitation of depth, 14 ft., of the present Welland Canal.

The Dominion of Canada is expending approximately $115 000 000 in the construction of the new ship canal to replace the present Welland Canal in order to permit the large bulk freight carriers of the four Upper Great Lakes to come to the eastern end of Lake Ontario. This will increase by nearly 200 miles the length of cheaper water haul in these vessels, and decrease correspondingly the distance over which freight must be carried by small vessels or by rail, at higher freight rates, to the seaboard.

The reason for the Canadian Government's heavy investment in the New Welland Ship Canal is shown by the following official statement:*

"It is expected that upon the completion of the New Welland there will be a reduction of about 2 cents a bushel in the freight rate on east-bound grain shipments, and that a large proportion of Canadian grain now being shipped by Buffalo and New York will be diverted to the St. Lawrence route".

In other words, through the New Welland Ship Canal, Canada expects to divert to the Port of Montreal much of the export grain trade now handled at the Port of New York. The rapid rise of Montreal as a port of export for grain, as contrasted with New York, is shown by Table 1.

TABLE 1.—Grain Exports from New York, N. Y., and Montreal, Que., Canada, in Bushels.

Year.	New York.	Montreal.
1924	105 738 000	152 069 655
1923	97 031 000	113 614 315
1922	127 438 000	148 667 983
1921	68 743 000	126 539 459
1920	83 101 000	57 897 357
1919	90 640 000	46 507 299
1918	63 391 000	62 484 000
1917	119 590 000	60 065 000
1916	153 961 000	70 688 000
1915	126 614 000	44 583 000
1914	61 844 000	75 891 000
1913	65 496 000	46 716 000
1912	53 523 000	38 811 000
1911	33 090 000	30 964 000
1910	17 898 000	27 968 000
1909	22 894 000	27 409 000

Prior to the World War, the Ports of New York and Montreal were about even in the quantity of grain exported. During the war years of 1915, 1916, and 1917, grain exports from New York were unusually heavy and much in excess of those from Montreal. Beginning with 1921, Montreal has exported each year a quantity substantially in excess of that exported from New York, the averages for the four years, 1921 to 1924, inclusive, being for Montreal, 135 223 000 bushels, and for New York, 99 738 000 bushels. In 1924, Montreal exported more than 152 000 000 bushels as against less than 106 000 000 bushels from New York, Montreal's excess over New York that year being about 44 per cent. Thus, Montreal has already surpassed New York as a port of grain export, and Canada expects through the New Welland Ship Canal to divert still more grain from New York to the leading port of the Dominion.

* Canada Year Book, issued by Dominion Bureau of Statistics, 1922–23, p. 657.

There are two factors that will enable New York to retain its share of the export grain trade. The first is the fact that under existing treaty regulations the Welland Ship Canal, like all other waterways of the Great Lakes, will be free alike to vessels of United States and Canadian registry. The second factor is that through the development of Oswego as a port for the accommodation of modern lake vessels, and the utilization of the Barge Canal route from Oswego to the Hudson River, the same savings on the transportation of ex-lake grain to New York may be effected as on transportation to Montreal. Thus, upon the opening of the Welland Ship Canal in 1930, the Barge Canal, coupled with the development of the Port of Oswego, will be of outstanding importance in safeguarding the export grain trade of the Port of New York, which is vitally needed to supply the bulk cargoes required for outbound vessels. Through the agency of the Barge Canal, New York will be able to maintain its present parity with Montreal as a port of grain shipment.

In connection with the need for development of the Port of Oswego by the Federal Government an extended investigation has been made of the commerce of the Great Lakes region and of the future traffic of the Barge Canal System. The conclusions reached are, briefly, that the Barge Canal route between Oswego and the Hudson River will probably carry, shortly after 1930, a traffic well in excess of 2 000 000 tons, or approximately equal to that of the entire Barge Canal System to-day. Furthermore, Barge Canal traffic to and from Buffalo will apparently be maintained at approximately its present tonnage figures, although there will be a decrease in the quantity of grain moved from Buffalo which should be offset by the transportation of other commodities. Shortly after 1930, therefore, it is to be expected that the total traffic of the Barge Canal System will be double the present traffic.

Not only will the importance of the Barge Canal to the Port of New York be greatly enhanced, but it will be a vitally important factor also in providing economical movement of freight between the new seaport at Albany and the Great Lakes region.

The people of the State of New York have builded wiser than they knew in providing the modern Barge Canal. It is only at the beginning of its career of useful service, not only to New York State, but to the whole nation. A ship canal from the Great Lakes to the sea is of doubtful economic value and in any case cannot be secured for many years. As Mr. Finch has suggested, the deepening and widening of the present Barge Canal may well afford the economical outlet to the sea which the people of the Middle West are now demanding.

MAURICE W. WILLIAMS,* M. AM. SOC. C. E.—The increase in traffic on the Barge Canal since its opening is not generally understood. In 1918 the canal was opened on its present line but not to full dimensions. It was not until 1919, therefore, that the canal can properly be said to have been fully opened. In the first year of its existence, 1918, the tonnage was approximately 1 160 000; in 1925, it was 2 344 000, an increase of 100% in 7 years.

* Cons. Engr., New York, N. Y.

The average progressive rate of increase of traffic in each year over the preceding year since 1918 has been about 10.65%, and is at least equal to the average rate of growth of similar comparable waterways. The annual rate of growth of traffic on the waterways of the State was only about 7% in the early Thirties, when the total tonnage was about equal to what it is now on the Barge Canal. If the factor of 10% be applied progressively to the traffic of 1925 for estimating the tonnage of the present and succeeding years, it is found that the canal will carry its nominal capacity of 20 000 000 tons per year in about 1948. The word "nominal" is used because nobody knows how much the canal can carry; but the Erie, Oswego, and Cayuga-Seneca Divisions certainly can carry 20 000 000 tons per year, to say nothing of the virtually independent Champlain Division.

One of the main commodity items, which the canal always has carried and probably always will carry, is grain. In 1925 grain formed about 40% of the total traffic. It is said in certain quarters that the exportable quantity of grain is bound to fall off so that in a few years there will be no grain to export. Such a conclusion is exceedingly questionable. The per capita production of wheat in the United States and Canada combined was about 7½ bushels per annum twenty-five years ago. To-day, it is about 10 bushels. In the last forty years the total production of wheat in the two countries has increased 50% faster than the population, and the wheat fields are still being pushed nearer the Arctic Circle. There is good reason for considering the grain of the United States and Canada jointly for purposes of export, in that the exportable grain of both countries moves through the ports of either with virtually equal facility. At present, there are exported annually from the ports between Montreal and Newport News, Va., inclusive, about 12 000 000 tons of grain per year, most of which is susceptible of being handled by the Barge Canal. Of this amount at least 7 000 000 tons move in the seven full months of canal navigation, to say nothing of the days in excess of seven months when the canal is open. There should be great quantities of grain to be moved for export as well as for domestic consumption for many years to come.

It has been brought out that the number of barges navigating the canal has not been increasing. To-day, there are slightly less than 800 barges. The number was about the same in 1924, but between 1922 and 1924 the average capacity of barge increased from 420 to 480 tons, or 15 per cent. During this period many old barges were scrapped and their places taken by new barges of much greater capacity. Sizes are still being increased, and some of the barges now almost fill the locks.

The speed of transportation by the Barge Canal compares favorably with that by rail. In October, 1925, the average car-mileage in this region was the maximum to date—slightly more than 33 per day. The report of the Superintendent of Public Works gives the mileage of the average barge navigating the Erie Division as 33.3 per day, virtually the same as by rail. The instance mentioned by Mr. Finch of the movement between Oswego and New York is in no way exceptional. Many other lines deliver nearly the same service and it may be expected that they always will.

There has been no decided movement of iron ore on the Barge Canal west from Lake Champlain. During 1925 and a year or two previously there was a movement of imported ore from New York to the Buffalo District and farther west; but it is too soon to make forecasts as to the development of this traffic. The movement of steel shapes and semi-manufactured iron from points near Buffalo eastward has developed in some quantity.

The average unit saving on freight actually transported on the canal is estimated to be about $1.15 per ton. That figure is obtained by consideration of the known saving on grain and on certain other commodities, the testimony regarding which was clear to the Barge Canal Survey Commission. The saving is the difference between what it actually costs to ship by canal and what it would have cost by the next least expensive route; the tonnage on which the saving is known is relatively large.

There are undoubtedly other advantages in the Barge Canal. It was also brought out in that testimony that the depression in freight rates in that section of the country due to the competitive influence of the Barge Canal effects an indirect saving to shippers amounting to $50 000 000, or more, each year. This figure is reached by a consideration of the tonnage produced and consumed in the area affected by the canal and a comparison of the transportation rates near to and far from the canal.

Undoubtedly the Barge Canal exercises a beneficent influence on the industry, commerce, and prosperity of the State of New York by assisting in maintaining the flow of commerce through the State. For example, it was brought out in the same testimony that if the Canal had not been available certain export grain would not have moved through New York State at all, but would have gone by some other route.

T. Kennard Thomson,* M. Am. Soc. C. E.—It is, of course, astounding to read that the best year's traffic on the New York State Barge Canal amounted to only one-third that carried on the old Erie Canal with its 7-ft. depth.

Two monumental blunders were made on the Erie Canal when it was recently reconstructed. The first was one of location in maintaining the hump of 57 ft. at Rome, which prevents any water from Lake Erie reaching the Hudson River. This requires the expensive storage basins at Delta and Hinckley to feed the canal east of Rome, as well as to provide the additional lockage.

The second error was in making it a Barge Canal with fixed bridges having a minimum clearance of 15½ ft. This was a political as well as an economic blunder made in the vain attempt to get the Cities of New York and Buffalo to vote for the enlargement. To realize this, it is only necessary to inspect the Canadian canals and note the use there made of the unrestricted height of boats, permitted by movable bridges.

The short-sightedness of New York City and Buffalo in thinking that the loss of a comparatively few stevedores who now load or unload freight at those two places could in any way compare with the prosperity of the entire State resulting from the free use of the canal, is simply marvellous.

* Cons. Engr., New York, N. Y.

New York City pays 25% of the tax of the United States, which means that it owns 25% of the total wealth and is therefore vitally interested in the prosperity of every foot of New York State, as well as the remainder of the Continent (Canada and Mexico included).

For the same reason, New York City and Buffalo will vastly benefit when the (Barge) Canal is used to its limit and when the Niagara and St. Lawrence Rivers carry ocean steamers to the Great Lakes, utilizing at the same time the hydro-electric energy of these rivers. This energy will amount to 8 000 000 h. p., saving 80 000 000 tons of coal per year, or $640 000 000—about $2 000 000 per day—most of which is now going to waste. A further result will be the construction of twenty Pittsburghs on the Canadian border.

When the speaker was one of the Board of five Consulting Engineers for the New York State Barge Canal, he was often asked "whether the Barge Canal would ever pay." His reply was that if the State of New York were to construct skyscrapers in New York City for the public, without equipping them with elevators, lights, water, etc., the public would never use them, as long as up-to-date fully equipped buildings were constructed by private capital near-by. Active agents would see to it that prospective tenants of the State-owned buildings were diverted to the privately owned structures.

For the same reason, when the State handles the Barge Canal as a business enterprise, going after the traffic, keeping the terminals, etc., up to date, the canal will be of enormous value to the State and to other parts of the country.

When the speaker hears estimates of future traffic for 20 to 50 years hence, he is forcibly reminded of a diary written by his father in 1842, describing a trip from Buffalo to New York by stage coach, in which he stated:

"Shortly there will be a railroad all the way from New York to Buffalo * * *. It was interesting to hear the discussion of the fellow passengers. The majority said that there would never be traffic enough to keep the rust off that one pair of rails, while the more enlightened ones thought that the single-track railroad would carry all the traffic between those cities for one hundred years to come."

Almost any one predicting the street traffic of automobiles and other vehicles in New York City, 10 or 15 years ago, would have been nearly as greatly in error. Who would have believed, 20 or 30 years ago, that the New York elevated and subway lines would carry the number of people they now do; and, second, that the 5-cent fare for this enormous number of people would involve a loss. Traffic estimates based on horse cars would be useless for modern conditions, and yet estimates are frequently made for conditions that are similarly radically different.

The Empire State does not need to worry about the possible volume of traffic—freight or human. All that it need do is to provide the facilities, and see that it gets its share, by exercising ordinary business common sense.

Harry Taylor,* M. Am. Soc. C. E.—It seems that the people of New York State and throughout the whole country have not really awakened to the value

* Maj.-Gen., U. S. Army (*Retired*), Washington, D. C.

of the Barge Canal. They speak of it as being more or less of a failure, as Mr. Finch has stated. The entire Middle West seems to be "sold" with the idea that a deep-water canal, either down the St. Lawrence River or across the State of New York, will solve all their troubles. They speak of tremendously cheap transportation on the Great Lakes and have an idea that this will be extended to the final port to which the grain is carried. Transportation on the Great Lakes is, presumably, the cheapest transportation in the world. There are a number of reasons for that; for example, the wonderful terminal facilities and the fact that after the boats start in the spring they make practically continuous voyages until they are laid up in the fall. There is practically no delay at the terminals.

The lake boats are also built, per ton of carrying capacity, very much cheaper than ocean steamers. Boats suitable for the Barge Canal can be built for less than lake boats and operated more cheaply. Why should not the lake boat bring commerce from the Upper Lakes to the Eastern ports at Buffalo, or better, at Oswego, transfer there to the barge, carry it across the State of New York, and there transfer it to the ocean steamer, either at Albany or New York. In that way the tonnage is carried in each boat by the cheapest possible carrier and the transfer charges would be more than saved.

E. P. Goodrich,* M. Am. Soc. C. E.—The author has pointed out with absolute accuracy that one of the principal handicaps under which the Barge Canal is now operating is the lack of boats. In this connection, it is apparent that the development of a port at Albany, where freight transfer can be made, will make it possible to shorten the total Barge Canal trip from Buffalo to New York. For many commodities this will be feasible, grain, for example, provided a grain elevator is established at the Albany port, as is required by the Board of Army Engineers. Under such circumstances, about 50% additional service can be secured from the existing boats so that the development of a port at Albany will be equivalent to that increase in the possibilities of Barge Canal operation.

F. Lavis,† M. Am. Soc. C. E. (by letter).—The author states that "there has never been a time when there was a more crying need for clear thinking in relation to waterways than at present." The writer agrees that there is certainly need of clear thinking at this time, but submits that there was just as much need twenty years ago, before the $175 000 000 which the Barge Canal cost was expended, and that clear thinking then would not only have saved this large sum for construction, but also the many millions more it has since cost in annual deficits.

Reference to the files of *Engineering News* for the period during which the then proposed bond issue was being discussed, will show that there were at least a few people, even at that time, who could think clearly and along lines of sound economics in regard to this matter, but their's were "voices crying in the wilderness", to which no heed was given. Let us therefore,

* Cons. Engr., New York, N. Y.

† Cons. Engr., Jersey City, N. J.

as Mr. Finch advocates, try and think clearly now before any more money is wasted on this or the proposed ship canal.

One of the principal difficulties in getting this clear thinking in considering waterways and water-power problems is the ingrained idea that is firmly rooted in the minds of probably 90% of the people of the United States that water is something provided by a Divine Providence and that man has only to stretch out his hand and take and use it as a free gift. Laymen, and even many engineers, ignore, or are unaware of the fact, that the development of electrical energy by water power and its transmission to the place where it may be used is often quite as expensive, sometimes more so, than the development of power by the combustion of fuel. So, also, do they think of waterways, ignoring the fact that artificial waterways often cost more to build and operate than railways or highways having equal capacity for traffic and capable of providing a far more flexible service. Unlike the canal-boat, a railroad car does not merely travel from a point on the main line of its home railroad to another point on that same main line, but it can and does travel from any side track at any point of production—mine, forest, or factory—to any other side track where the material it carries is needed. The writer has no knowledge of the relative cost of the actual power required to haul a ton on rails as compared with a ton on the water, but it hardly seems likely that the cost is less on the water than on the railroads.

Mr. Finch states that one of the reasons why the canal has not been used to a greater extent is because there has not been and even now is not an adequate supply of boats. He draws or attempts to draw, an analogy between this and a railroad without rolling stock. A railroad at the start provides rolling stock for the business in sight and increases its rolling stock as the business develops, and only as it develops. The fact that after eight years of operation, the demand for traffic on the Barge Canal is only about 10% of its capacity is surely evidence that there is no crying need for this adjunct to the State's transportation facilities. It seems inconceivable, if transportation by this means was desirable, was economically sound, was a convenience, or was, on the whole, cheaper, that the many commercial enterprises in this country, which are ever on the alert to save a few cents, to say nothing of dollars, would not have availed themselves long ago of the opportunity which the canal is supposed to offer.

Even with the canal waterway offered free to users, the State carrying all the expense of maintenance and operation as well as interest and amortization on the original investment, its value and utility is so little apparent that its use is really negligible, and its importance as a factor in transportation almost nil.

Mr. Maurice W. Williams has stated that "the net expense of the Barge Canal has been estimated to be $9 864 763.55 annually until the average date of maturity of the bonds." Inasmuch as the canal was opened in 1918 and the date of maturity of the bonds is 1939, the net expense, which in the writer's opinion is a loss to the people of the State, is somewhat more than $200 000 000, in addition to the original investment of $175 000 000.

The argument is advanced that the losses on the Barge Canal are offset by savings to shippers by reason of the lower freight rates on the railroads which are maintained by reason of the water competition. Of course, the fallacy of this is evident not only because freight rates on railroads are pretty closely regulated by the Interstate Commerce Commission, but because, in any event, looking at the matter from the broad standpoint of the whole National or even State economy, and in view of the fact that the railroads are owned by the public and by the shippers, savings or expenditures on them are for account of the people as a whole.

The investment, therefore, of $175 000 000 and annual expenditures totaling $205 000 000 more of the people's money to save money for certain shippers at the expense of the people who have invested in the railroads, hardly seems sound economy.

In the early days of canals, when the railroads were operated as unregulated and almost unrestricted monopolies, there might have been some force in an argument of savings by reason of the water competition, but it is not sound in view of the conditions under which the railroads are operated to-day. It is wise therefore, as Mr. Finch suggests, to try and think clearly in regard to the economics of this question. This requires some consideration of the economics of transportation, the principal factor in which is the railroad system of this country. It should be remembered that the railroads to-day are under strict and, on the whole, quite efficient regulation by the National Government, as well as by the individual States.

The cost of transportation by the railroads is now based closely on actual costs of operation and maintenance, plus a very moderate rate of interest (6%) on the investment in the property. The amount of this investment is also fixed by the Government and represents a sum which is undoubtedly not greater than the reproduction cost of the properties as of the time the various parts were constructed, and less than their reproduction cost to-day.

On the whole, the money invested in the railroads is the money of the people and, in the long run, any gain or loss is a gain or loss to the nation as as a whole. Transportation by railroad is so vital to the continuing life of the country, to say nothing of its continuing prosperity, that the railroads must be kept at a high state of efficiency if the nation is to continue to live and prosper.

There is of course no argument to be advanced against the development of waterways *per se*, but when they are developed, as was the Barge Canal, solely on the basis of offering competition to the railroads, and when this competition results in a loss to the people of the State of more than $370 000 000, such expenditures are to be deplored.

The construction of any artificial waterway can usually be justified only if, after taking into consideration annual charges for interest and amortization on the investment, costs of maintenance and operation of the waterway, and costs of operation of the vehicles traveling over it, it produces ton-miles at rates comparable with or less than rates at which these same or similar ton-miles can be produced by other means. This, as far as the writer knows,

has not yet been accomplished on any inland waterway in the United States, the Great Lakes not being included, of course.

One trouble with clear thinking in this matter is that so many people are still living and thinking in the manner of the days when the waterways were as vital factors in the life of the nation as railways and highways are to-day. They have not yet realized that because of extreme flexibility as to location, time, and character, and because of adaptability of vehicles, the highways and railways are rendering services so far superior to those which these inland waterways can possibly render that, except in a very few instances, these latter have little place in the modern plan of economic transportation.

J. K. Finch,* Assoc. M. Am. Soc. C. E. (by letter).—This clear brief statement of the problem of the New York State Barge Canal by the State Engineer and Surveyor brings before the Society for discussion a problem in engineering economics of the first magnitude.

In the writer's opinion too many great public undertakings are settled pro or con by popular appeals based on pride, patriotism, or politics. The public is too often called on to decide questions which actually demand expert and unbiased study by economic and engineering experts. Kipling's village decided by popular vote that the world is flat. Has not this been done with the Barge Canal?

In many cases engineers themselves have carefully avoided the economic issues involved and have been content to act simply as agents in carrying out the structural requirements of undertakings which have been decided on by others. One of the greatest engineers, Wellington, pointed out clearly that the engineer's duties went beyond mere design and required him to advise his client whether the project in general was economically sound. Indeed, the economic factors of most great engineering problems are so intimately tied up with the technical engineering possibilities and questions involved, that a man must have engineering training as well as a thorough understanding of economic principles to handle them.

Engineering schools have been very backward in recognizing this fact. In general, they have devoted their energies to perfecting their students in the latest advances in science and their applications, and developments in the technique of engineering. This has been one of their fundamental duties; but there can be no question that the reason why engineers have been adverse to giving attention to the broader economic problems of their professional work is due to the fact that they realize their lack of the training and background necessary to handle such questions. The writer ventures to express the opinion that the engineering schools and the professional societies should turn their attention to this field and that in this phase of their activities modern engineers can be of still greater usefulness to their country and their clients. From this standpoint the paper in question is an important contribution to engineering activities and merits the fullest possible discussion.

* Associate Prof., Civ. Eng., Columbia Univ., New York, N. Y.

Mr. Finch has outlined the history of the Barge Canal. Briefly, it shows that for fifty years "Clinton's ditch" was a great economic success. In the beginning it had practically no competition, and for a time it competed successfully with the young railroads, which, however, cut more and more into its traffic. Then came the Civil War, and, with the closing of the Mississippi, both the canal and the railroads had all that they could handle. By 1872, however, the decline in the canal business began. The Erie Canal went through the same experience as most of the other canals of the country. Like the others, the Erie had its day of great economic usefulness, but, due to the fact that canals were slow means of moving freight, that they were only available during part of the year on account of ice conditions, and in general, due to the expansion of the country, they became only parts of through systems and therefore costly transshipment was necessary. They could not deliver freight to the consumer's door as was often possible through railroad sidings, and because of the increasing efficiency and effectiveness of railroad operation, the Erie, like other canals, became practically an abandoned waterway.

At an early date in its history it had been saddled with so-called feeder canals which proved to be financial burdens. At various times it was widened and deepened and, until its last years, these efforts at increasing its usefulness had been productive investments. Therefore, in spite of the fact that 3 000 of the 3 750 miles of canals built in the United States had been abandoned—because, like the Erie, they had outlived their usefulness—an agitation was started in the Nineties to again widen and deepen the Erie, and make it into a Barge Canal, with the idea that this would again prolong its life.

In 1903 "the people of the State of New York" decided to undertake this work, which has cost them to date more than $178 000 000 (omitting interest and maintenance charges which are not earned and amount to about $7 000 000 per year). The result of this expert opinion on the part of the public has been on a par with that of Kipling's village already mentioned. The Barge Canal now carries about one-tenth of the traffic it could handle, and it has been estimated* that it would cost "the people of the State of New York" less to pay the railroads to handle this traffic than it does for them to operate the canal.

The arguments advanced in 1903 urging this expenditure were based largely on appeals to the pride of "the people of New York State." The Erie Canal made the Empire State; the Barge Canal will retain for this State its economic supremacy. More than 70% of the population of New York, that is 7% of the population of the United States, live within a 30-min. walk of the canal. This means that the products of 8 000 000 people need and should have a cheap means of transportation, such as the Barge Canal will afford, etc. These, of course, are mere statements and not the kind of sound economic truths that can be backed up with operating results.

It was obvious that traffic originating within the State could not be expected to be sufficient in volume to justify the canal. The failure of the

* See Report of the Bureau of Railroad Economics, Washington, D. C., June, 1925.

feeder system of earlier years showed this. The population along the canal, therefore, is of no particular interest. Mr. A. Barton Hepburn, however, has clearly stated the principal source of traffic hoped for, namely, Western grain. In a book,* primarily devoted to an attempt to justify the Barge Canal enlargement, he argued for several pages that the Barge Canal would bring back to New York the grain export business which was rapidly being taken away from it by Canadian facilities, thus giving the City of Montreal the export supremacy formerly held by New York City. Later, in one paragraph, he demolishes this argument with the statement that "the growth of our population has shattered this hope, for it is likely in the future we will consume at home practically all the grain we raise." Typical of the spirit of the arguments advanced at that time, he goes on, "our enlarged waterway must and will find other means of justifying its existence." He does not state what they are and "the people" are still looking for them.

A most thorough study of the inland waterway problem was made by Mr. Charles Whiting Baker in 1920.† He concluded that most of the inland waterways had served their purpose and could not compete with modern means of transportation. These studies are worthy of careful attention in connection with this problem.

At present, therefore, "the people of New York State" are saddled with the biggest "white elephant" ever purchased, the most colossal engineering failure of modern times, and the question is what to do about it. An attempt has been made to turn it over to the Federal Government as a possible basis for a ship canal. The War Department engineers apparently do not take to this idea. Mr. Finch now brings this problem before the Society. He suggests that the State has not done all that it should to make the Barge Canal useful. A few years ago it lacked terminal facilities, but in spite of the fact that "terminals have been constructed at every city and nearly every village along the line of the canal," traffic goes elsewhere.

It is now stated that one of the reasons why the canal has failed to get traffic is the lack of boats for transporting this traffic. It is suggested that the canal be again widened and deepened and that the State go into the navigation business, either directly or by financial aid. While it is true that the Barge Canal has not had all possible chances to make it a success, is it not also true that if the traffic was there to justify a larger canal equipment there would be private capital ready and anxious to go into the canal transportation business?

Barge Canal traffic must come from two sources: (*a*) within the State; that is, local traffic along the line; or (*b*) through traffic. While the writer does not pose as an expert in these matters he would like to ask three questions.

1.—Is it not true that local traffic can never be expected to justify the New York State Barge Canal?

* "Artificial Waterways of the World", N. Y., McMillan, 1914.

† "The Future of Inland Water Transportation," a series of articles in *Engineering News-Record*, Vol. 84.

2.—Is it not true that the only type of through traffic in sight and in sufficient volume to justify the canal is the export grain business?

3.—Is it reasonable to suppose that the New York State Barge Canal, merely a possible link in a through route and hence involving transshipment, can successfully compete with Canadian through routes in handling export grain, which admittedly originates almost entirely in Canadian territory?

There are sound economic reasons why New York is losing her grain business. The writer knows of no sound reasons why the canal will be, or can be made in the near future, an economic success.

It has been argued that the Barge Canal is needed to keep railroad rates down to reasonable figures. This is equivalent to stating that the commission form of control for privately owned public utilities is a failure. It assumes that effective regulation of transportation requires the construction of competing lines at huge costs, lines which would do little or no business but are necessary to protect the public from exorbitant charges. Such a policy would, of course, result in economic disaster. Government ownership would be preferable to this, and experiments to date along these lines show conclusively that in most cases such ownership is simply another name for political mismanagement which, unlike private management, cannot be controlled and is both dangerous and costly.

Is it not best to reduce the maintenance costs of the Barge Canal to a minimum until a time arrives when traffic requirements can be based on something better than "hope"; to put capital cost down to profit and loss; and to look on the New York State Barge Canal as a great lesson purchased by "the people of New York State" at no less than one-half the cost of the Panama Canal, for the purpose of proving that great engineering undertakings are only successful when based on sound economic reasoning and advice?

E. E. King,* Assoc. M. Am. Soc. C. E. (by letter).—In comparing costs by different systems of transportation, it is necessary to consider in any shipment the cost from the door of the shipper to the door of the consignee. In the case of small packages shipped by freight there is probably little difference in the cost of transportation whether the shipment is on the railroad or on the canal-boat. The costs of carting the packages to and from the freight depot and the wharf would in all likelihood not differ a great deal. The cost of moving from one station to another would probably be a little less by boat than by train. The cost of terminal expense may be relatively so high in both cases that the difference in cost of line haul may not be a factor of serious consideration. Regularity of service might probably be the chief matter of concern in the case. Both railroads and canals should devise systems of handling package freight at terminals in order to reduce the cost of shipment.

Package shipments in less than carload lots constitute, however, only a very small percentage of all freight business. The great bulk of railroad deliveries is in carload shipments. Where both the consignor and consignee have water-front facilities they could no doubt ship more cheaply by water

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than by rail; but not many industries are so situated. Coal, grain, and ore-handling seem to have been generally planned for such transportation in the Great Lakes region. Grain loaded from cars or lake boats at Buffalo, through elevators into barges on the canal and delivered through elevators at New York City into ocean-going ships could, no doubt, be handled more cheaply by the canal than by the railroad; and for that reason, most of the canal traffic will continue to be grain and similar bulk material.

An off-canal shipper can ordinarily find it just as cheap, or often cheaper, to ship by rail in the case of carload lots if the industry has its own siding and the expense of team haul is thus eliminated. The expense of terminal haul by truck or team is often an item of considerable magnitude compared with the line haul. It frequently happens that the expense of collecting and delivering the freight at the terminals is more than the entire line haul.

Frank L. Bolton,* Assoc. M. Am. Soc. C. E. (by letter).—For several years the writer has been hoping that the problems confronting the New York State Barge Canal would be opened for discussion before the Society. The true facts concerning the canal have been most difficult to ascertain as a result of so much erroneous information being in circulation. All those who have the interest of the canal at heart, will welcome this paper, coming from one who knows whereof he speaks.

Furthermore, at this time, when there is an awakening interest in internal waterways, the paper is particularly opportune. The author's history of the New York State canals and his description of the physical features of the Barge Canal, enables one to visualize the growth over 110 years, from a "ditch", 4 ft. deep and 28 ft. wide, to the present magnificent waterway, about 800 miles long (with connecting rivers and lakes) and costing the State of New York $175 000 000.

The writer hardly realized that 70% of the population of the State of New York lives within about 2 miles of the Canal System. This fact puts the canals in an economic position of great importance and indicates that, when the waterway is functioning properly, the benefits therefrom will be widespread and effective upon a very large proportion of the people of the State.

The author has pointed out that there are not enough boats on the canal at the present time and concludes that "this lack of carriers is the main reason why the Barge Canal is not transporting the tonnage for which it was designed and to-day is capable of carrying", giving several reasons.

In this viewpoint the writer fully concurs. There are, however, other factors of more importance that have discouraged boat operation. They are so fundamental that, if properly recognized, they will answer the question of "What is wrong with the canal?"

Before proceeding to answer this question, consider what can be said in favor of the canal, as it is to-day.

* Ithaca, N. Y.

The writer is offering this discussion from a dual viewpoint, first as an engineer interested in water transportation in general and in the physical features of the Barge Canal in particular; and, second, as a shipper of freight, who is now using the canal in a satisfactory way as a dependable, economical means of transportation.

From a physical standpoint, the canal appears to be a practical working waterway. Without doubt there are some changes and improvements necessary but these minor defects can be corrected without any large expenditure of money by the State. As an illustration, the condition at the outlet of Cayuga Lake may be mentioned. At this point the lock walls (Mud Lock) are so constructed that when Cayuga Lake is at its maximum stage, the water is about a foot higher than the top of the lock walls and it is impossible to lock boats through. This defect in design can be remedied by raising the lock walls and the lower lock-gate 1 ft. or 2 ft. During the past few seasons of navigation, there have been delays at this point because the boats could not be locked through until the lake was drawn down or until a temporary dam had been constructed along the top of the lock walls.

As to the depth of water in the canal, about which so much erroneous information has appeared, the writer has, during the past two seasons of navigation (1925-26), loaded boats to a 10.5-ft. draft. These boats have moved without difficulty from a point on Cayuga Lake six miles north of Ithaca, to New York City, as well as westward to Tonawanda and Niagara Falls.

Having shipped 100 large barges loaded with salt, averaging from 500 to 700 tons in capacity, and having found that the movements of these boats proceeded in a dependable manner, running on a satisfactory schedule, the writer cannot see that any pertinent criticism of the physical conditions of the canal can be raised. It is evident, therefore, that an answer to the question as to what is wrong must be found elsewhere.

Obviously, capital must be interested in canal operation, if a sufficient number of boats are to be forthcoming for handling the available tonnage. The problem of interesting capital to-day is far different from the situation in the old days of operation on the Erie Canal. When the boats operating on the canal had a carrying capacity of only 200 or 300 tons, or less, the motive power was horse or mule teams. An individual with small capital could finance the construction of one or more small boats and with a team of mules could tow them along the canal and make a living. The boat owner usually made his home on his boat both summer and winter and with small expense of operation the canal thus offered an easy line of independent endeavor for many individuals. Under these conditions, many "Canalers" entered the trade with their own small equipment and as a result ample boats and much competition was available for the canal.

To-day, with the enlarged canal, the tow-path is gone and with it most of the small boats and the individual operators. Those desiring to enter into canal transportation must obtain large amounts of capital in order to secure larger and more modern boats, expensive towing equipment, etc. As the author has pointed out there are single-motor ships operating on the canal

to-day, that cost $175 000 per unit. Here is the difficulty in the present situation, capital has not been interested in a large way in going into canal transportation. If it can be shown how capital can be interested, and that the undertaking will be profitable, it is believed that the problem of the canal will be solved.

In addition to the reasons pointed out by the author, as to why capital has not been interested, the writer wishes to add another of vital importance. This is the matter of a definite continuing policy for the canal, as well as a permanent personnel for its operation and maintenance. The writer hopes that the author, in his closing discussion, will enlarge on the question of a continuing policy.

The canals are operated by, and under, the administration of the Department of Public Works of the State of New York. This Department has many other duties besides looking after the Barge Canal. The State Highway Department, the Department of Public Buildings, and the duties of the State Engineer and Surveyor all come under it. Inasmuch as a large portion of this Department's work is highway work, and as many of its engineers are highway engineers, it is logical that the highway point of view predominates in the operation of the canal. The attitude of the State, as expressed through the Department of Public Works, seems to be this: "Here is the canal ready to be used, please come and make use of it". Obviously, this attitude will not sell the canal to those people who can use it, as they want to know whether the conditions now obtaining as to physical features as well as the favorable attitude of the present operating personnel will persist over a long enough period of years to justify their operations on the waterway. No criticism is intended on the present operating personnel; the writer's point of view is that the canal does not have a permanent personnel engaged solely in advancing the interests of the waterway.

As now organized, those in authority, having to do with the policy and operation of the canal, change with every change in the State Administration, or more often, and this militates against a continuing policy. The difficulty of those using the canal in having to deal with a constantly changing personnel may be illustrated by referring to the defect in the design of the lock walls at the outlet of Cayuga Lake mentioned previously. This defect was called to the attention of the State Engineer several years ago, but before steps were taken to remedy it, he passed out of office. The matter was again called to the attention of the newly elected State Engineer some months ago, and as his term of office has expired, it will probably be necessary to take the matter up again with another official, in order to get a minor engineering defect corrected. With a continuing personnel, shippers would be doing business all the time with the same individuals and would have a much better chance of getting results.

A continuing policy, as well as a continuing personnel, must be provided, if this splendid waterway is to develop properly the tonnage for which it was designed and which appears to be available. It would seem that the State of New York, having spent $175 000 000 for this Canal System, would see fit to place this large investment in the hands of a permanent board or

authority, to be operated on a business basis and in much the same way that a railroad would be operated had it a capital investment of $175 000 000.

The Governor of the State of New York has advocated a quasi-public organization, to be known as a "Water Power Authority" for handling the water-power situation in the State. Why not a "Canal Authority", organized along similar lines, for administering this investment of $175 000 000?

Various suggestions have been made from time to time by those interested in some phase of canal transportation, looking toward further expenditures by the State for docks, terminals, tipples, freight-packet service, and similar items, with the thought that these things are needed for making the Barge Canal more useful. The writer is opposed to any such expenditures. These matters will be taken care of and should be taken care of by private capital. If assurance can be given that the physical conditions of the canal will be maintained over a period of years and that a continuing policy and organization will be provided, it appears inevitable that capital will become interested in the canal, as positive economic advantages to shippers of freight and to operating companies can be shown by past experience.

The writer cannot agree that enlarging the present canal would prove a greater inducement to capital to place boats in operation. The suggestion was made that deepening the canal from the present 12 ft. to 15 ft. and increasing the width of the channel to 110 ft., would allow self-propelled barges of more than 2 000 tons capacity to operate efficiently and the carrying capacity of the larger barges to be increased 20 to 30% with practically no increase in operating expenses. With the possible exception of grain traffic, the writer can see no economic advantage in being able to operate single units of 2 000 tons capacity. For general use on the canal a unit of 2 000 tons of cargo is larger than can economically be handled by the shippers and receivers of freight. A fleet of four barges of 500 tons capacity each is a much more flexible arrangement for practically all commodities, except possibly grain. Looking to the future, it is the writer's belief that as the tonnage on the canal develops, the grain movement will constitute a lesser percentage of the total than at present. One of the problems facing the shippers of many commodities is the inability of the consignee to take in and handle at one time as much as 500 to 700 tons of freight. Furthermore, the smaller units now in use on the canal enable the shipper to dispatch a fleet of several units and distribute them along the canal at different points without tying up the motive power during the time required for discharging cargo at local points.

It is true that the cost of operating on the canal is reduced rapidly as the tonnage handled at one time increases, but it is not necessary for low costs that the tonnage handled at one time be contained in one bottom. The author cites one carrier, that operates a fleet of five barges and a steamer utilizing the full capacity of the locks on a draft of 10 ft. 4 in., with a carrying capacity of 3 600 tons. With such an operation now possible, the writer cannot see where any greater inducement to capital would be offered by enlarging the canal to such a point that 2 000 tons capacity could be handled in one bottom

The present Barge Canal can and will justify itself as a dependable waterway independent of, and without consideration for, any deep waterway that may be constructed from the Great Lakes to the Atlantic. It will of necessity be many years before the proposed deep waterway can be put into operation even if construction should be initiated within the next few years; and after the completion of such a deep waterway, the present Barge Canal, if handled properly, will become an important feeder for the ship canal as well as serving locally the territory along its course. The question as to what will happen to certain tonnage on the Barge Canal at the end of ten to fifteen years, when a ship canal might be in operation, has little to do with arriving at a proper solution of the Barge Canal problems at this time.

Recognition should be made of the fact that a considerable tonnage is now being handled over the Barge Canal by shippers along its route, and that these shippers have made investments in the way of docks, storage warehouses, and other items of plant and equipment necessary for shipping and receiving freight over the canal. These investments have been made in good faith with the belief that the State would get its waterways in such shape as to be continually useful. These shippers must be considered pioneers, who are pointing the way to a broader and more general use of the State's waterways and, as such, are entitled to a definite and fixed policy on the part of the State concerning the present and future status of the operation of its waterways.

Roy G. Finch,* M. Am. Soc. C. E. (by letter).—The paper was prepared primarily for the purpose of presenting the facts relating to the New York State Barge Canal. The writer is grateful for the discussion which brought out such widely divergent views as to the past, present, and future of this waterway.

Because this canal is now used to only one-tenth of its capacity, it does not necessarily follow that the engineers, who twenty-five years ago approved of a "barge" type of canal to connect the Great Lakes with tide-water, failed to analyze the problem correctly or that they "blundered" in selecting this type as best adapted to serve as the waterway connection across New York State. On the contrary, what definite proof has been presented to show the economic advantage of attempting to operate large vessels on restricted inland channels as against the plan of operating on ocean, lake, and canal the particular types of craft that can operate on each waterway most efficiently? The latter was the governing economic principle laid down in 1903, which dictated the selection of the "barge" type of canal. Up to the present time it has not had a fair chance to prove or disprove the soundness of such principle. Apparently from the discussion of General Taylor he still feels that this type of canal can be made to serve adequately its original intended purpose—a feasible economic waterway connection between the Great Lakes and tide-water.

Mr. Thomson refers to "two monumental blunders" in constructing the Barge Canal, namely, the introduction of the summit level at Rome and the

* Cons. Engr., Albany, N. Y.

building of fixed bridges. It seems doubtful whether tonnage has been driven away from the canal due to the existence of this summit level. If Mr. Thomson had in mind the adoption of the plan embraced in one of the earlier studies of a route for a deep waterway, it would seem that this plan had its disadvantages. It contemplated one level from Fulton, on the Oswego River, to Frankfort, on the Mohawk River, a distance of 72 miles. It called for raising the level of Oneida Lake and making a cut of 60 ft. in the vicinity of Rome. The topography and the extent to which this section of New York State has been built up and developed, would make a canal constructed on such a plan objectionable, particularly so when the available water supply is sufficient to avoid limiting the project to such a plan.

The "second error" to which Mr. Thomson refers has to do with the bridges, which have a minimum clearance of 15½ ft. above the water. He points to the Canadian canals, where the use of boats of any height is made possible by movable bridges.

It must be kept in mind that the country through which the Canadian canals are constructed is quite different as to density of population and industrial development, from the highly developed part of New York State traversed by its canal system. With any great amount of traffic, the required frequent operation of the many railroad and highway bridges might prove highly detrimental to the successful handling of the constantly increasing volume of rail and vehicular traffic in this section of the State. Furthermore, the carrying capacity of the general type of craft navigating the Canadian canals is about 2 600 tons. On the New York State Canal System, with restricted bridge clearances, a barge fleet consisting of five barges and a steamer can carry 3 600 tons. If the canal were deepened so that large self-propelled units could safely load to a draft of 11 ft. 9 in., they would then have a carrying capacity of at least 2 000 tons. This would seem to indicate that boats, properly designed and equipped and comparing favorably as to carrying capacities with the types of craft in use on the Canadian canals, could be operated successfully even with limited bridge clearances.

Professor Finch asks three pertinent questions. The first is whether local traffic can ever be expected to justify the canal. It seems to be the consensus of opinion that the canal is adapted for packet service. Packet lines, however, have not been organized and operated to the extent of furnishing any sound basis for an estimate as to the amount of local tonnage that might be attracted to the canal. Due to the advent and mobility of the motor truck, it would seem doubtful whether local traffic in sufficient volume can be secured to "justify" the New York State Barge Canal.

His second question, "Is it not true that the only type of through traffic in sight and in sufficient volume to justify the canal is the export grain business?" can probably be answered in the affirmative with certain reservations. Grain is the commodity now moved in greatest tonnage on the canal, followed by stone, sand, and gravel, which, in turn, is followed by petroleum products. Prior to the opening of the canal, the anticipated grain movement, which is an eastbound movement, was the basis of the estimate that 70% of the total

freight movement would be eastbound. It is interesting to note that in 1926 the westbound movement was about 51% of the total freight movement on the canal.

The writer believes that General Taylor in his discussion answered the third question asked by Professor Finch.

The discussion by Mr. Bolton is worthy of the most careful consideration, for he speaks not only as an engineer, but as a shipper "who is using the canal in a satisfactory way as a dependable economical means of transportation." He questions whether the deepening of the canal to 15 ft. and increasing the width in restricted sections to 110 ft. would result in placing more boats in operation. The locks were originally designed with widths of 28 ft., which was in harmony with a bottom width of channel in restricted sections (75 ft.) controlling for less than 30% of the length of the entire system. The width of the locks was subsequently increased to 45 ft., but with no corresponding increase in channel dimensions. Under existing conditions, the canal is "over-locked". The abutments of the bridges spanning restricted sections are so located that channel improvements can be carried out and not interfere with these structures. In connection with the maintenance dredging, the channel is gradually being deepened to 14 ft.

There is a wide difference of opinion among boat operators as to what is the most efficient type of operating unit. Boats having lengths of 100 ft. and widths of 20 ft. are competing with self-propelled units with lengths up to 258 ft. and widths of 42 ft. If boats of this latter type prove to be the most efficient unit, their successful operation demands a wider channel in restricted sections and the opportunity to load to the maximum depth permissible and clear the miter sills of the locks. Certainly the channel improvements suggested would not be detrimental to the operation of the smaller units and would make it possible for the larger carriers to increase their carrying capacities, which should result in increased earnings and thus make canal navigation a more attractive field for investment. Until it is demonstrated that boats can be operated at a profit, capital will continue to be reluctant to enter the canal navigation field; and a canal without an adequate number of boats will never be a success.

Mr. Bolton, in discussing a continuing canal personnel and the adoption of a definite and fixed policy on the part of the State concerning the present and future status of the operation of its waterways, has touched on a vital point so far as the future of the canals is concerned. As the result of a Constitutional amendment, a re-organized State Government started functioning on January 1, 1927, and all the activities of the State are now placed in a limited number of departments, which number cannot be increased unless a further Constitutional amendment is approved. The canals are placed under the jurisdiction of the Department of Public Works and the head of that Department is appointed by the Governor and the term of office is contemporaneous with that of the Governor. There seems to be little likelihood at this time of placing the canals under a separate department or commission.

During the past few years, certain changes have been made with a view to securing a more permanent canal-operating personnel by placing more posi-

tions under Civil Service. This should result in an increased continuity of service among those charged with the maintenance and upkeep of the canal. The effect is bound to be helpful.

Whether justified or not, there has been a feeling that the State might "scrap" its canal system or turn it over to the Federal Government as a part of a ship canal project. Such a sentiment has retarded the development of the canal and has been detrimental to the building up of its traffic.

In the writer's opinion, the greatest stimulating influence which at this time could be brought to bear on the canal would be for the State, in no uncertain terms, to re-affirm the policy of continuing the present system; to maintain it at the highest state of efficiency; and to make changes and betterments designed to promote the best interests of navigation and thereby give to this waterway a fair chance to prove whether or not it can serve as a dependable, ample, and economic water connection between the Great Lakes and the Atlantic seaboard.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1649

QUANTITIES OF MATERIALS AND COSTS PER SQUARE FOOT OF FLOOR FOR HIGHWAY AND ELECTRIC-RAILWAY LONG-SPAN SUSPENSION BRIDGES*

By J. A. L. Waddell,† M. Am. Soc. C. E.

With Discussion by Messrs. T. Kennard Thomson, H. B. Muckleston, Allston Dana, I. Oesterblom, Leon S. Moisseiff, Francis P. Witmer, V. A. Eberly, Horatio P. Van Cleve, Wendell P. Brown, William G. Grove, M. Godard, O. H. Ammann, Frank W. Skinner, and J. A. L. Waddell.

Synopsis

The ground covered by this paper is thoroughly and concisely explained by its title, which indicates even its limitations. Its object is to enable any engineer of good, general experience to compute, in two or three hours, when full preliminary data are furnished, the total cost of any projected highway or electric-railway or combined-highway-and-electric-railway suspension bridge and its approaches, including all the items of expense which, under ordinary conditions, would form the grand total cash outlay necessitated by the contemplated enterprise.

The paper is divided as follows into six distinct parts:

Introduction;
Assumed Data;
Findings;
Applications;
Deductions; and
Conclusion.

Under "Assumed Data" the following topics are listed and discussed more or less at length: (1) Roadway; (2) Live Loads; (3) Impact; (4) Wind

* Presented at the meeting of December 1, 1926.
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Loads; (5) Floors; (6) Towers; (7) Cables; (8) Back-Stays; (9) Pier Shafts; (10) Pier Bases; (11) Anchorages; (12) Tower Encasement; (13) Lighting; (14) Materials; (15) Intensities of Working Stresses; (16) Unit Prices; and (17) Panel Lengths, Truss Depths, and Cable Sags.

Under the heading "Findings" are given by diagram, and discussed, the quantities of materials per square foot of floor in: (*a*) Flooring; (*b*) Floor Systems and Hand-Rails; (*c*) Lateral Systems; (*d*) Stiffening Trusses; (*e*) Suspenders; (*f*) Main Cables; (*g*) Back-Stays; (*h*) Towers; (*i*) Pier Shafts; (*j*) Pier Bases; (*k*) Tower Encasement; (*l*) Lighting; (*m*) Anchorages; (*n*) Costs; and (*o*) Total Amounts of Metal.

The "Findings" show likewise by diagrams the quantities of all materials in steel trestle approaches to highway and electric-railway bridges of all kinds. Under this heading are treated also certain "Miscellaneous Costs" that cannot be included on the basis of "per square foot of floor," such as retaining walls, earth embankments, plazas, right of way, and property damages.

Under the heading "Applications," in order to demonstrate how the information furnished can be utilized, a specific case is worked out, all the necessary preliminary data being listed under the three divisions: (*p*) General; (*q*) Dimensions; and (*r*) Financial.

The computations made from these data are presented in the three succeeding groupings: (*s*) Cost Table for Main Span; (*t*) Cost of Approaches; and (*u*) Grand Total Cost of Project.

Under the heading "Deductions" there are discussed the suspending of the floor of side spans from the back-stays instead of using independent steel trestle approaches, the comparative costs of these two types of layout, and the determination of the extreme feasible lengths of suspension spans.

In the "Conclusion" the most important point raised is the writer's appeal to his brother engineers, not only to discuss this paper but also in their discussions to bring up and treat any other new matters of interest connected with suspension-bridge design and construction, in order to add materially to the general knowledge of the Engineering Profession concerning this important sub-division of the specialty of bridgework.

Introduction

In America, due to the wonderful development of the automobile, and to its use by all classes of people, especially in touring, highway bridges have become even more important than railway bridges. Moreover, on account of the exceedingly heavy motor trucks now used, both the live load and the dead load for the new structures are far greater than those in vogue a decade or two ago. The building of new post roads necessitates the bridging of the largest rivers of North America; and some of these are either so wide or so important for navigation as to require long spans.

As shown by comparative computations,* the limiting economic span length for highway cantilever bridges, pure and simple, as compared with the corresponding suspension bridges, was 1 000 ft. in 1918. Some engineers contend that it is shorter and others that it is longer; hence, probably in most cases this is not far from correct. However, the present falling price of steel wire for cables tends to lessen this span length of equal cost, as does also the adoption of the deflection method of computing moments and shears in the stiffening trusses, instead of the elastic method formerly used. For this reason that length has been chosen as the inferior limit for this investigation, the superior trial limit being the 5 000-ft. span. The curves, however, have been extended by extrapolation down to spans of 500 ft., and one of them up to a length of 7 000 ft.

Certain proposed river crossings demand spans well in excess of 1 000 ft., necessitating suspension bridges. Among others may be mentioned the one over San Francisco Harbor to connect San Francisco with Oakland and Berkeley, Calif., one to cross the Hudson River, connecting New York, N. Y., at 179th Street with a number of important municipalities in New Jersey; and one across The Narrows of New York Harbor to join Brooklyn, N. Y., and Staten Island.

As there are almost no reliable data concerning the costs of such long spans, the wildest kinds of guesses are constantly being made by laymen, and even by engineers. This condition has led the writer to make the present investigation.

During the last few years he has had occasion in his practice to compute in detail the costs of six proposed highway suspension bridges with span lengths of 1 550 ft., 1 750 ft., 1 975 ft., 2 500 ft., 2 800 ft., and 3 200 ft., in addition to numerous other computations.† Unfortunately, the widths of the decks and the live loads for these six bridges do not agree; consequently, it was necessary to make the computations methodically *de novo* and to use the results of all previous calculations merely for checking.

Assumed Data

Roadway.—For the sake of rigidity in both the span and the towers, the horizontal distance between the central planes of the trusses should be not less than one-thirty-fifth of the span. On this basis, adopting a clear roadway of 80 ft., with no sidewalks, for spans up to 3 300 ft., the width increases gradually thereafter to 129 ft., for a span of 5 000 ft.

No consideration has been given to the matter of sidewalks; for, without appreciable error, their cost per square foot may be computed as one-half of that for the main roadway. For instance, in the case of a bridge having an 80-ft. clear roadway and two sidewalks, each 10 ft. wide in the clear, the calculations should be made for an area of 90 sq. ft. per lin. ft. of span.

The spacing of the electric-railway tracks was taken as 10 ft., from center to center, or eight lines of track for an 80-ft. clear roadway carrying tracks only.

* "Economics of Bridgework," by J. A. L. Waddell.

† *Loc. cit.*, Chapter XIII.

Live Loads.—For the deck and the suspenders the live loads adopted were taken from the writer's treatise, "Bridge Engineering", being Class A (with a 20-ton truck load, however) for the highway; Class C for the sidewalks; Class 30 for electric-railway stringers; and Class 25 for electric-railway cross-girders.

The live loads for stiffening trusses, cables, towers, and pier tops are shown in Fig. 1.

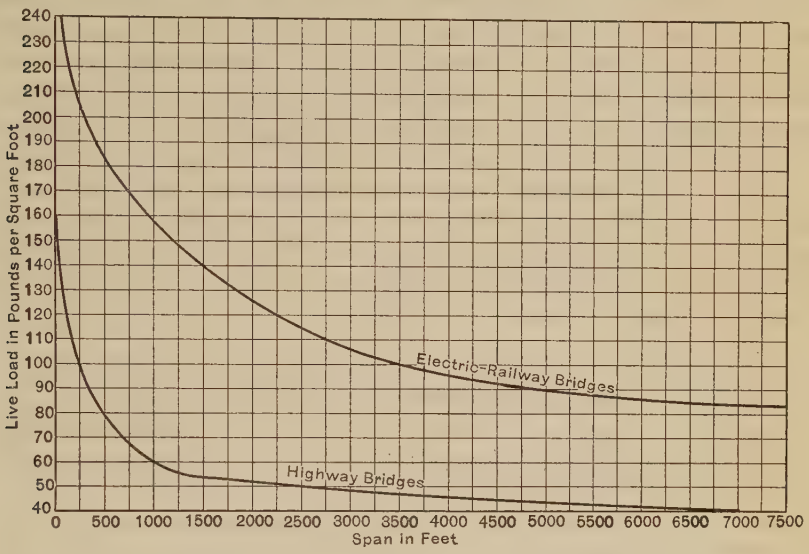

FIG. 1.—LIVE LOADS FOR SUSPENSION BRIDGES.

Impact.—The impact allowances used on floors and suspenders in both highway and electric-railway bridges are those given in Chapter VII of "Bridge Engineering" for highway structures. This assumption is valid, because recent investigations have proved that the impact in electric-railway bridges is far less than in the corresponding steam-railway structures, and, as nearly as may be, identical with that for the corresponding highway bridges. Owing to the great lengths of all trusses and cables, the impact upon them has been assumed to be zero.

Wind Loads.—The unit wind loads adopted were 30 lb. per sq. ft. for spans of 1 000 ft., reducing gradually to about 20 lb. per sq. ft. for those of 5 000 ft. The exposed area of vehicles was not considered for highway bridges, but for electric-railway bridges two lines of cars (one not in any way protecting the other) were assumed to be subject to wind pressure with 10 sq. ft. of exposed area per linear foot of track, reasonable lengths of trains being used.

Fig. 2 records the wind pressures per linear foot of span for both highway and electric-railway suspension bridges. It will be noticed that for each type of bridge there are two lines of loading, the upper one being for the structure as a whole and the lower one for the trusses and laterals. Modern studies in suspension-bridge designing have shown that a part of the wind load is taken

up by the cables and carried by them to the tops of the towers, and that the proportion thus carried increases with the span. That proportion is assumed to be a function of the quotient of the distance between central planes of trusses by the span. This assumption is probably only approximately correct; but, in view of the present state of professional knowledge, it will have to suffice. It accounts for the horizontality of the lines in Fig. 2 beyond the span of 3 300 ft., where the width of structure begins to increase. Theoretically, there should be a slight upward tendency in these parts of the lines to correspond with the upward inclinations of the graphs for the total wind load on the structure; but such a refinement is not warranted, as it is claimed by certain authorities that in long spans the effect of wind pressure on trusses and laterals is hardly worthy of consideration, owing primarily to the massiveness of the cables and also to the rigidity of the wide concrete platform composing the highway deck. Be this as it may, it was not considered wise to cut down the effective wind loads on trusses and laterals any more than is indicated in Fig. 2, notwithstanding that it is never possible for great live loads and great wind loads to act simultaneously on any highway bridge, and seldom, if ever, on any electric-railway bridge.

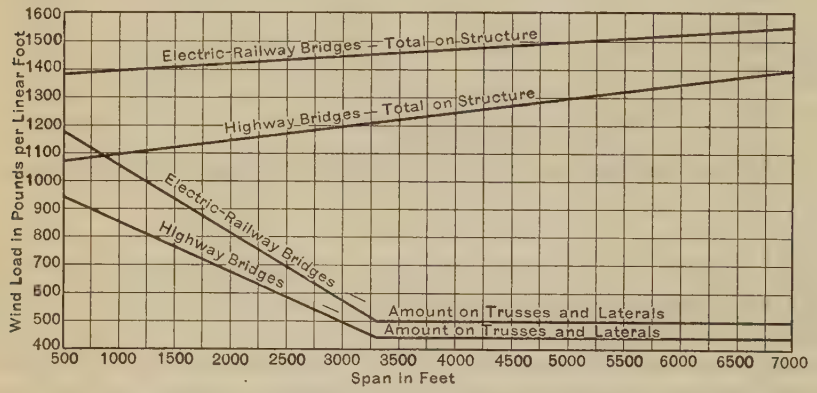

FIG. 2.—WIND LOADS FOR SUSPENSION BRIDGES.

Floors.—In long-span highway bridges it is economical to reduce the thicknesses and weights of the floor-slab and pavement to a minimum and thus the amount of metal throughout the structure (except in the stiffening trusses, which are not affected to any material extent by the dead load). This may be accomplished effectively in three ways, namely, by using metal liberally for reinforcement; by adopting a light aggregate for the concrete; and by making the pavement as thin as is consistent with due provision against its failure under heavy traffic.

In the Philadelphia-Camden Suspension Bridge, the use of a special system of reinforcement reduced the thickness of the base to 6 in.; and that is about the limit of feasibility, unless the spacing of the stringers be made ultra-close. In these computations a similar slab, 6½ in. thick, was adopted with 5-ft. stringer spacing.

During the World War, the Federal Government in building reinforced concrete ships used concrete weighing only from 110 to 120 lb. per cu. ft. There is also a light aggregate, known as "Haydite", made of clay or shale burned and aerated by a patented process that reduces the weight of plain concrete to 106 lb. per cu. ft. It is claimed that such concrete is stronger and more impervious to water than that composed of the ordinary aggregates; but these assertions need corroborative evidence. In making these computations, however, the writer has assumed that a satisfactory light aggregate will be available for building the floors of future suspension bridges.

In respect to bridge pavements, engineers are fast coming to the conclusion that a great thickness is not really necessary, but that a thin pavement of bituminous concrete can be used and replaced as it wears off. In these calculations a thickness of 1½ in. is assumed.

The electric-railway flooring adopted consists of the usual wooden ties and guard-rails and 90-lb. steel rails.

Towers.—The towers are flat at their bases and are rigidly attached to the masonry of the piers, the tops of which are about 25 ft. below clearance elevation to permit the feet of the columns to be spread transversely to the structure. Their dimensions are to be such as will permit their tops to spring properly in the longitudinal direction of the bridge under all possible variations of temperature and loading. Theoretically, it would be more scientific to hinge them at the bottom. This would require less metal, but would involve some trouble and expense in staying them during erection.

Cables.—The cables are assumed to be of the best procurable quality of carbon-steel wire. Certain engineers are of the opinion that eye-bar cables are preferable. It is evident, however,* that no untreated eye-bars can compete with wire for cables, except perhaps in very short spans, and that heat-treated bars might be more economical than wire for fairly long spans. A few years ago heat treatment for eye-bars was in an experimental stage, and wire cables were very expensive. Now heat treatment is claimed to have been well developed but, on the other hand, the cost of wire cables has been greatly reduced, and is still falling. Hence, until the prices of these materials have reached a condition of comparatively stable equilibrium, the economy of eye-bars over cables for any long-span suspension bridge is questionable.

Back-Stays.—For the purpose of this investigation the back-stays are assumed to be unloaded and to have the same inclination to the vertical as the main cable has at the tower top. Their lengths were made sufficient to bring their ends down to an elevation of 20 ft. above low water.

Pier Shafts.—The shafts of the piers are to be of plain concrete, extending down to an elevation 2 or 3 ft. below low water, where they will rest either on rock or on a base of timber and concrete.

Pier Bases.—When a shallow rock foundation is not available, the pier shafts are assumed to rest on pneumatic caissons sunk to bed-rock at a depth

* "Comparative Economics of Wire-Cables and High-Alloy-Steel Eye-Bar Cables for Long-Span Suspension-Bridge," *Proceedings,* Engrs. Soc. of Western Pennsylvania, Vol. 36, 1920–21, p. 418.

of about 100 ft. below low water—in other words, the bases are made 100 ft. high. This is likely to be their height in most cases, for the reason that, with bed-rock sloping down rapidly from the shore, the pier will probably be located, for the sake of economy, as far out as practicable; and the assumption made involves the maximum safe air pressure.

Anchorages.—Two types of anchorages have been provided, namely (*a*) those resting in the dry on bare bed-rock; and (*b*) those supported on fairly stiff clay, reinforced by long piles, and encased in a box 30 ft. deep filled with concrete.

For each of these types it is feasible to construct cost curves that will give comparatively accurate results. There is, however, still another case, that is, when the anchorage is obtained by tunneling into a hillside of solid rock; here the governing conditions will vary so greatly that it is not practicable to plot the costs of the anchorages. However, no great error would be made in assuming the same cost as for anchorages on bare rock. The structures have been computed on the assumptions of solid masses of concrete, of no attempt at decoration, and of cheap material to obtain weight inexpensively. If the anchorages are pierced by arched openings, if a stone masonry covering be used, or if other decorative features be adopted, the costs given by the diagrams will have to be materially increased.

In designing the bases of the anchorages, no reliance was placed on the horizontal resistance of the piles, and the center of pressure was always kept within the middle third of the base. Each anchorage was made in two independent parts, one for each side of the structure. For anchorages on pile foundations, in plan each part is wide in front and narrow behind; and in elevation it is low in front and high behind.

Tower Encasement.—The tower encasement adopted consists of a shell of reinforced concrete surrounding, but never touching, the steel tower. It gives the appearance of a solid masonry shaft of a decidedly decorative character. Such a method of improving the esthetics of the bridge is legitimate, but, of course, by no means necessary.

Lighting.—An adequate system of electric lighting for each structure has been assumed—of a decorative nature to conform properly to the importance of the proposed bridge.

Materials.—For comparatively short stringers, rolled carbon-steel I-beams were assumed; but for the longer ones nickel-steel built-beams were adopted. That alloy was used also for the cross-girders and the stiffening trusses. In the lateral system the diagonals were generally computed as nickel steel; but carbon steel was used in the over-head bracing wherever mass, for rigidity rather than for mere strength, was the prime requisite.

In the suspenders and their connecting details nickel steel was used, and for the cables wire rope of the greatest strength attainable. For the towers silicon steel was adopted, because *per se* it is generally more economical than nickel steel, the great advantage of the latter being in the reduction of dead load—a consideration that is negligible in towers.

Intensities of Working Stresses.—The unit stresses used for proportioning steel members were as follows:

Carbon steel in tension	16 000 lb.
Silicon steel in tension	24 000 "
Nickel steel in tension	28 000 "
Wire cables in tension	70 000 "

For compression members the following formulas for allowable stress were used:

Carbon steel	$16\,000 - 60\,\frac{l}{r}$
Silicon steel	$24\,000 - 90\,\frac{l}{r}$
Nickel steel	$28\,000 - 105\,\frac{l}{r}$

For combinations of live load, dead load, and wind load, an increase of 30% in these intensities was permitted.

Unit Prices.—The schedule prices adopted for materials in place when computing the costs of suspension bridges per square foot of floor were as follows:

Carbon steel	$0.07 per lb.
Silicon steel in towers	0.09 " "
Nickel steel	$0.10 per lb.
Wire cables	0.18 " "
Floor-slab, including reinforcement	35.00 " cu. yd.
Pavement	3.00 " sq. yd.
Electric-railway track	6.00 " lin. ft.
Shafts of piers	20.00 " cu. yd.
Shafts of anchorages	15.00 " cu. yd.
Bases of pneumatic piers	35.00 " cu. yd.
Bases of pile anchorages	30.00 " cu. yd.
Piles below anchorage bases	1.25 " lin. ft.

TABLE 1.—Assumed Dimensions of Suspension Bridges.

Span length, in feet.	Panel Length, in Feet.		Truss Depth, in Feet.		Sag of Cable, in Feet.	
	Highway.	Railway.	Highway.	Railway.	Highway.	Railway.
1 000	24.4	25	24	25	100	105
2 000	33.9	40	34	40	175	185
3 000	44.1	54.7	44	55	250	265
4 000	54.1	70.2	54	70	325	345
5 000	64.1	84.8	64	85	400	425

Panel Lengths, Truss Depths, and Cable Sags.—Table 1 gives the assumed panel lengths, truss depths, and cable sags for the five span lengths on which the calculations of quantities of materials were made.

The fact that the panel length in every case has been made about equal to the truss depth is merely for convenience in computing, and is by no means a requisite; nor are the panel lengths themselves of any great importance, in so far as the weight of metal in the floor system is concerned, as will be shown.

Again, in case the panel lengths for the long spans for any reason appear to be too great, they can be divided by the adoption of sub-diagonals and intermediate verticals.

The quantities of materials and their costs are sub-divided as follows: (1) Flooring, including pavement; (2) metal in floor system and hand railings; (3) metal in lateral system; (4) metal in stiffening trusses; (5) metal in suspenders; (6) metal in main cables; (7) metal in back-stays; (8) metal in towers; (9) lighting; (10) pier shafts; (11) pier bases; (12) anchorages; and (13) tower encasement.

Findings

Flooring.—For each square foot of highway deck there will be required 0.54 cu. ft. of reinforced concrete and 0.125 cu. ft. of pavement, weighing in combination 80 lb. The similar quantities for the electric-railway deck will be 5.5 ft. B. M. of timber and 6.6 lb. of rails and attachments, weighing in combination 30 lb.

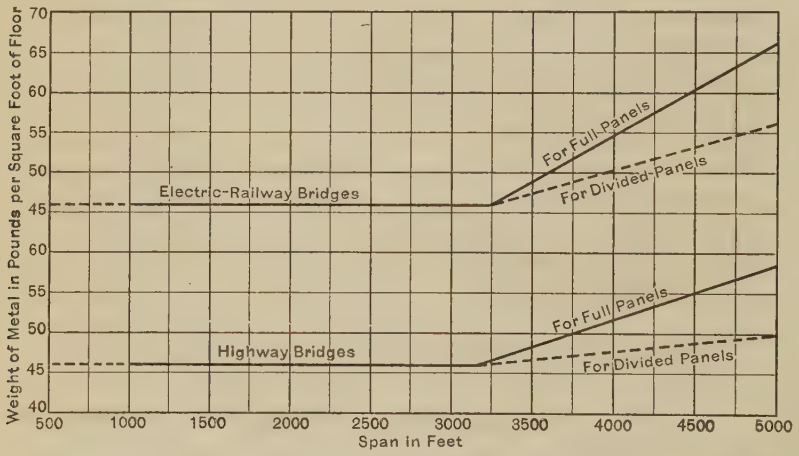

FIG. 3.—WEIGHTS OF METAL IN FLOOR SYSTEMS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

Floor Systems and Hand-Rails.—To find the quantities of metal in the floor systems ten cases were estimated, for panels of different lengths in both highway and electric-railway structures. Strange to say, up to spans of 3 300 ft.—the point at which the width of structure was increased—there was almost no variation from 46 lb. per sq. ft. of floor, regardless of either panel length or character of loading. The metal was nickel steel for spans of 2 000 ft. and more; one-half carbon and one-half nickel steel in spans of less length. Fig. 3 shows the results for both highway and electric-railway bridges.

A valuable deduction may be drawn from these data as to the increase in weight of metal with an increase in the perpendicular distance between trusses.

If N is the percentage of increase in the width of the structure between trusses, X the corresponding percentage of increase in the weight of metal per square foot of floor for the floor system, and A is a constant, then:

$$X = A\,N \qquad (1)$$

For highway bridges the value of A is about 0.5 and for electric-railway bridges about 0.8.

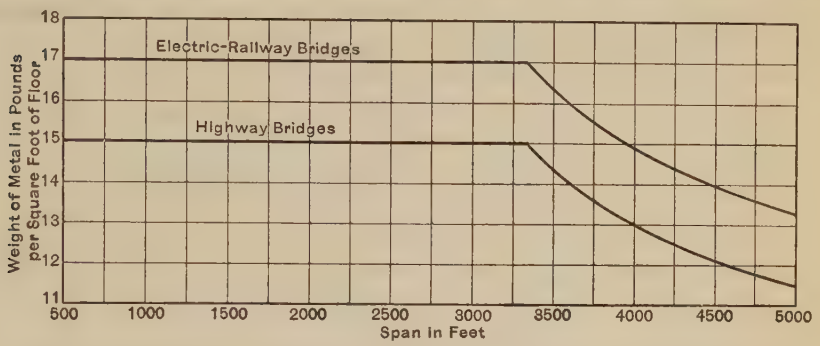

FIG. 4.—WEIGHT OF NICKEL STEEL IN LATERAL SYSTEMS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

Lateral Systems.—In Fig. 4 are recorded the weights of metal per square foot of floor found for the lateral systems of both highway and electric-railway suspension bridges. In spans up to 2 000 or 2 500 ft., certain members of the lateral system may be built of carbon steel; but most of the weight recorded is that of nickel steel.

For short spans the weights may appear to be too high, especially when compared with those for the Philadelphia-Camden Bridge. The explanation is that in all the writer's designs he has used over-head bracing, as, on general principles, he is opposed to pony trusses, especially in long spans. It is true that the tension in the suspenders tends to keep the top chords of pony trusses from getting greatly out of line; but, when computing the chord sections for compression, it would be difficult to determine satisfactorily the proper value of the slenderness ratio.

The peculiar peaks in Fig. 4 are due to the sudden widening of the structure, beginning at the span of 3 300 ft.; for, although the weight of metal per linear foot is increased slightly by the widening, the weight per square foot of floor is diminished.

Stiffening Trusses.—In Fig. 5 are indicated the weights of nickel steel for the stiffening trusses of both highway and electric-railway suspension bridges.

It will be noticed that there are two curves of weights for each of these types of structure. The upper one was computed by means of formulas based on the "elastic theory",* and the lower one was deduced from the upper one

* "Economics of Bridgework," Chapter XXIX.

by the application of a reducing coefficient* (Fig. 6), computed by Leon S. Moisseiff, M. Am. Soc. C. E. In discussing the Philadelphia-Camden Bridge, Mr. Moisseiff shows that for the stiffening trusses of the main span, 1 750 ft. in length, the deflection theory reduces the weight of metal to 64% theoretically, or 67% actually, of that found by the elastic theory, the difference in these percentages being due to the use of certain minimum sections in designing. Although this reason would hold good in a span of 1 750 ft., it would not apply to a span of 5 000 ft. It was this fact and the writer's judgment that induced him to assume the percentage as 64 for this longer span. An accurate analysis by the deflection theory probably would show a somewhat smaller percentage than 64, but the purpose of this paper did not warrant the trouble of verifying this surmise.

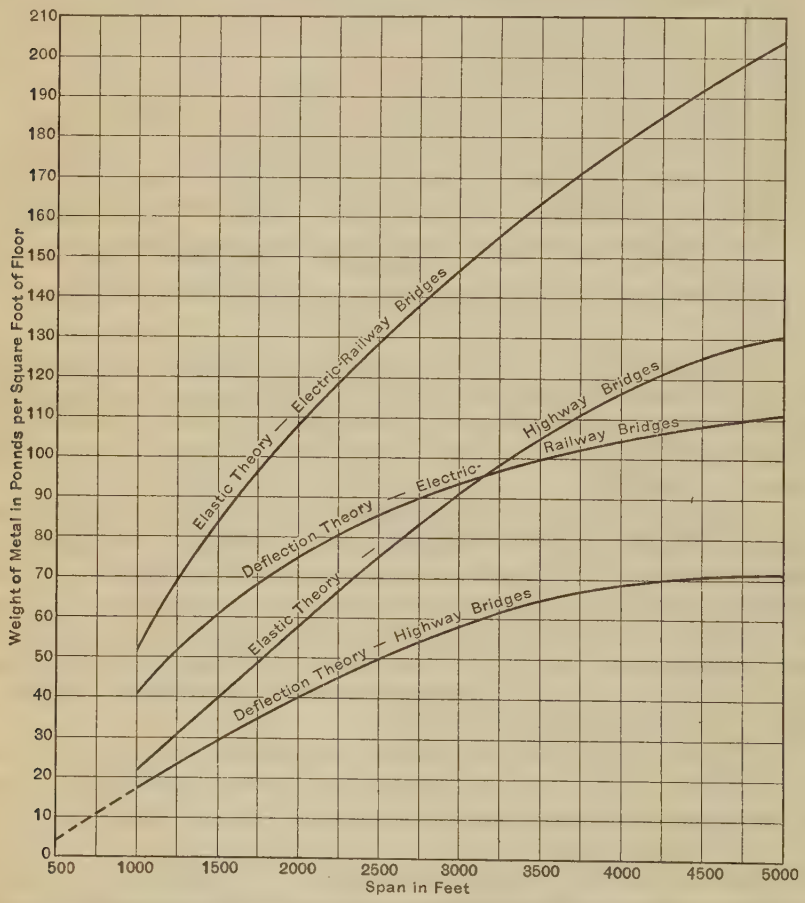
Weight of Metal in Pounds per Square Foot of Floor
Span in Feet
Elastic Theory — Electric-Railway Bridges
Deflection Theory — Electric-Railway Bridges
Elastic Theory — Highway Bridges
Deflection Theory — Highway Bridges

FIG. 5.—WEIGHT OF NICKEL STEEL IN STIFFENING TRUSSES OF SUSPENSION BRIDGES.

Another assumption is that although the ratios for weight reduction found by Mr. Moisseiff in the case of the Philadelphia-Camden Bridge, in which

* "The Towers, Cables, and Stiffening Trusses of the Bridge over the Delaware River, between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

the quotient of truss depth by span is 0.016, are correct for all span lengths where that quotient prevails, they would not hold good for other proportions. If c is the coefficient given by Fig. 6, and q is the quotient of truss depth by span for the case under consideration, then the reducing ratio, r, would be given by the equation,

$$r = 1 - (1 - c) \times \frac{0.016}{q}$$

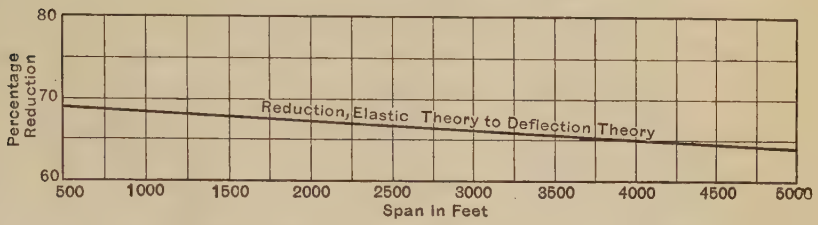

FIG. 6.—PERCENTAGES OF REDUCTION IN WEIGHTS OF METAL IN STIFFENING TRUSSES OF SUSPENSION BRIDGES AS FOUND BY THE ELASTIC THEORY TO AGREE WITH THOSE FOUND BY THE DEFLECTION THEORY.

For instance, in the case of a 4 000-ft. highway span, Fig. 6 gives a reducing ratio of 0.65, while the quotient of truss depth by span is 0.0135; hence the reducing ratio used for this case was:

$$r = 1 - (1 - 0.6_5) \times \frac{0.016}{0.0135} = 0.585$$

In favor of ignoring the effect of the live-load stress reversal, previously alluded to, there is much to be said:

First.—The latest long-span suspension bridges have been thus designed, consequently the pace has been set for bridge designers to follow, unless they feel that a mistake has been made in so doing.

Second.—When the greatest live load is on a highway bridge, it is generally distributed almost uniformly, in which case no reversion will exist; in an electric-railway structure the stress reversal is much more probable.

Third.—When stress reversal does occur, there is sure to be a considerable interval of time between reversals, even under electric-railway loading, hence the metal will have an opportunity to recover before being stressed in the opposite direction.

On the other hand, modern designing of stiffening trusses has a single trend, namely, toward cutting down sectional areas; and such economizing may be carried too far. It must not be forgotten that some of the truss members of the Brooklyn Bridge have crumpled under live load without the simultaneous existence of any wind pressure worth mentioning; hence, before an engineer starts the actual designing of any long-span suspension bridge, he should make some computations to determine the actual unit stresses in those members of that old structure which were crippled, using the modern theories for finding stresses, and, as nearly as practicable, the live load per linear foot that caused the crippling.

Wire Cables and Towers.—In Figs. 7 to 11 are given the quantities in suspenders, cables, back-stays, towers, and pier-shafts. Fig. 9, referring to

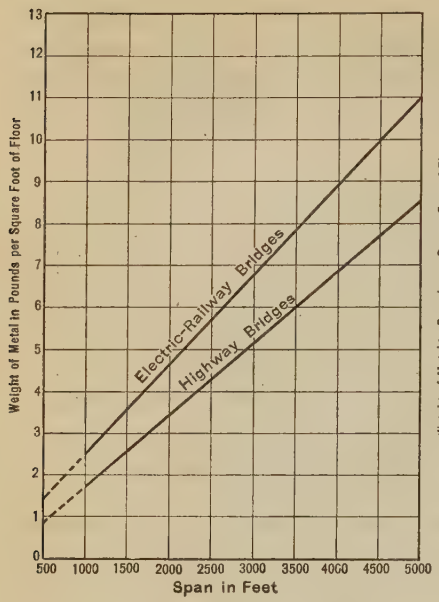

FIG. 7.—WEIGHTS OF NICKEL STEEL IN SUSPENDERS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

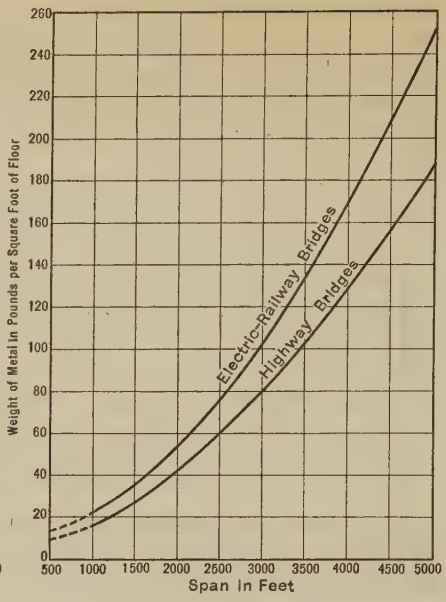

FIG. 8.—WEIGHTS OF WIRE AND WRAPPING IN MAIN CABLES OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

the back-stays, is based on the assumption that they carry no vertical load other than their own weight. When the back-stays do carry vertical loads by means of suspenders, this diagram will not be needed, as Fig. 8 only should then be used.

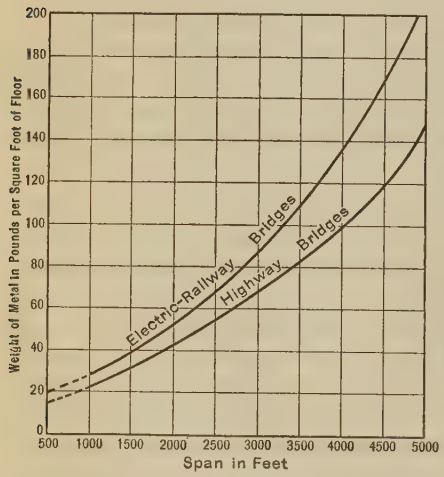

FIG. 9.—WEIGHTS OF WIRE AND WRAPPING IN BACK-STAYS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

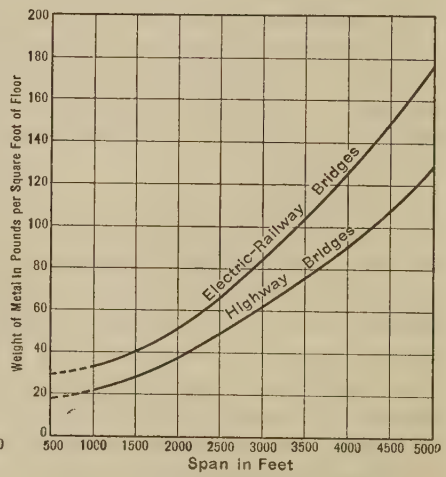

FIG. 10.—WEIGHTS OF SILICON STEEL IN TOWERS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

Pier Bases.—Fig. 12 gives the volumes of bases for caissons of piers. As the assumed height of base is just 100 ft., it is easy to compute by proportion the corrected volume for any other depth of foundation.

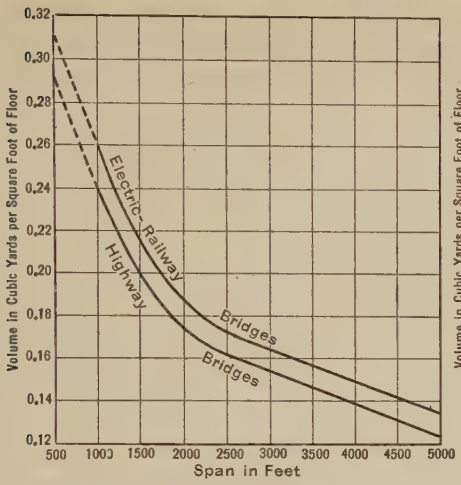

FIG. 11.—CONCRETE IN SHAFTS OF PIERS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

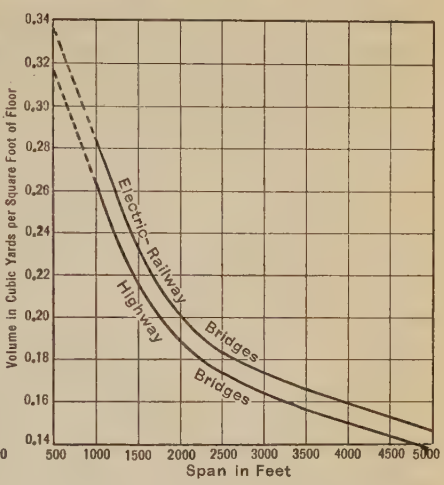

FIG. 12.—BASES OF PNEUMATIC OR OPEN DREDGING PIERS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

Figs. 11 and 12 were plotted on the assumption that there were no side spans hung from the back-stays, thus throwing the cost of the two piers in with that of the main span. If, however, the side spans are supported by the back-stays, the total cost of the main piers will not be increased, as the loadings on the pier tops are practically the same for the two cases; but the quantities given on the diagrams will have to be multiplied by the ratio (less than unity) of the length of the main span to the total length of main and side spans (generally, but not always, equal to twice the length of the main span).

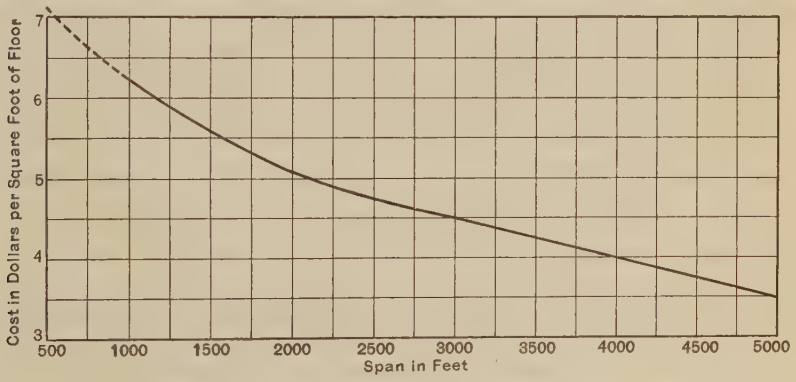

FIG. 13.—COST OF TOWER ENCASEMENT FOR HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

Tower Encasement.—The cost of tower encasement (Fig. 13) will vary materially with the artistic and the economic ideas of the designer. The diagram is based on the architectural features for a long-span suspension-bridge

study made by the writer, which features were suggested in consultation by Theodore H. Skinner, M. Am. Soc. C. E.

As in the case of the pier shafts and pier bases, the cost of encasement for two towers is thrown entirely on the main span of the structure. It should be noted that this cost is almost the same for both highway and electric-railway suspension bridges, the latter involving slightly more expense because of a small difference in tower heights.

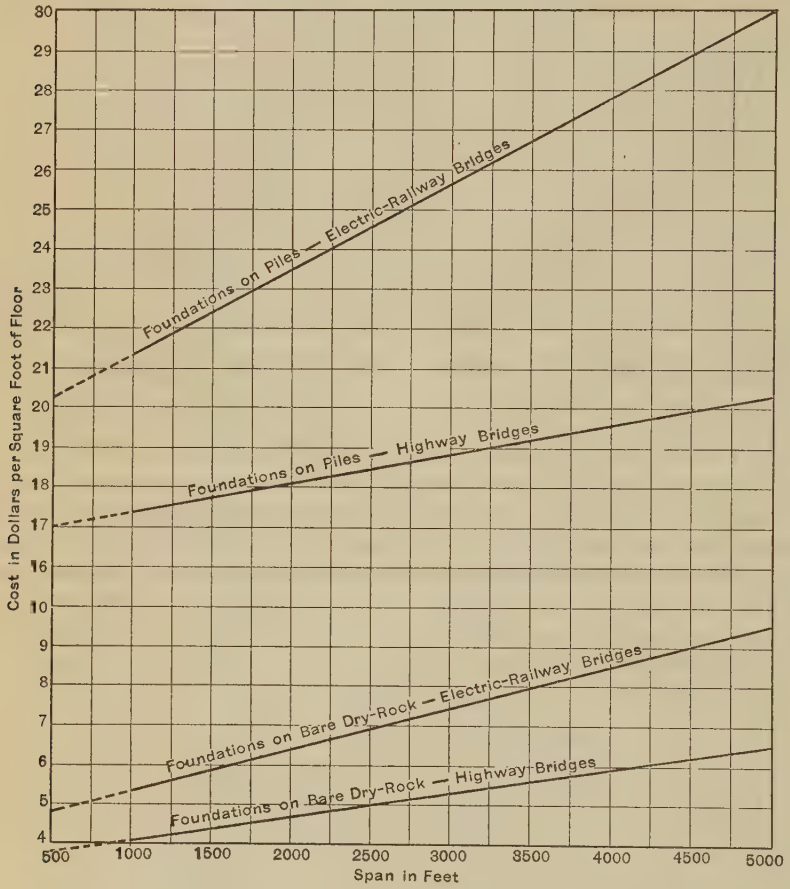

FIG. 14.—COSTS OF ANCHORAGES FOR HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

Lighting.—The cost of lighting is an essential item in estimating any bridge of importance. It is independent of the span and varies almost directly as the total area of the deck. Its amount, however, is small, being only $0.10 per sq. ft. of floor for a simple lighting system and possibly twice as much for an elaborate one. These prices include the cost of conduits for the electric wires.

Anchorages.—In Fig. 14 are recorded the costs for two typical kinds of anchorage for each of the two classes of bridges. The lower curves will suffice

approximately for the case of anchorage by tunneling into hillsides of hard rock.

Costs.—Fig. 15 shows the total costs per square foot of floor for both highway and electric-railway suspension bridges, each type having anchorages resting on dry rock and on piles. As the tower encasement is just as likely to be omitted as adopted, its cost has not been included in this diagram.

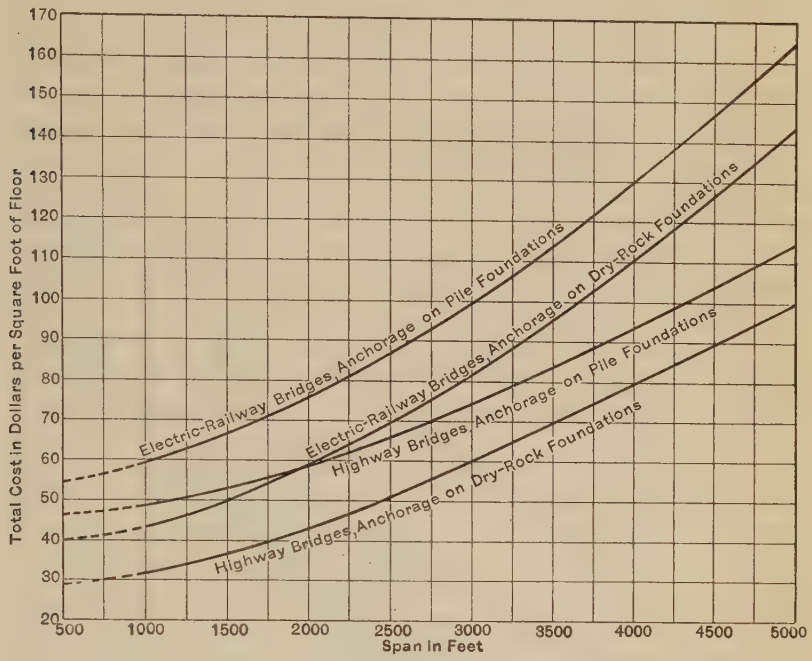

FIG. 15.—TOTAL COSTS OF HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

Total Amounts of Metal.—To answer the question "How much steel is required" for any particular suspension bridge, Figs. 16 and 17 have been prepared. Fig. 16 gives the weights of structural steel and of wire-cable steel for suspension bridges, in which the side spans are not hung from the back-stays. Fig. 17 gives the corresponding weights when the back-stays carry the side spans.

The diagrams, Figs. 1 to 17, furnish all the information required for computing the cost of any ordinary highway, electric-railway, or combined highway-and-electric-railway suspension bridge that may be contemplated, excluding the approaches. To make this paper as complete and serviceable as possible, certain diagrams (Figs. 18, 19, and 20) for materials in highway bridge approaches taken from the writer's office records, hitherto unpublished, will be included.

Steel trestle construction, here assumed, is the cheapest kind of approach to a bridge; hence, if reinforced concrete, or arches, or any unusual or fanciful kind of construction be used, the approaches will cost considerably more than these three diagrams indicate.

In a highway steel trestle the roadway pavement, with its supporting reinforced concrete base, costs, including the reinforcement, about \$1.00 per sq. ft. of floor; the open deck of an electric-railway trestle costs about \$0.70 per sq. ft. of floor; and the footwalk slabs cost about \$0.50 per sq. ft. of floor.

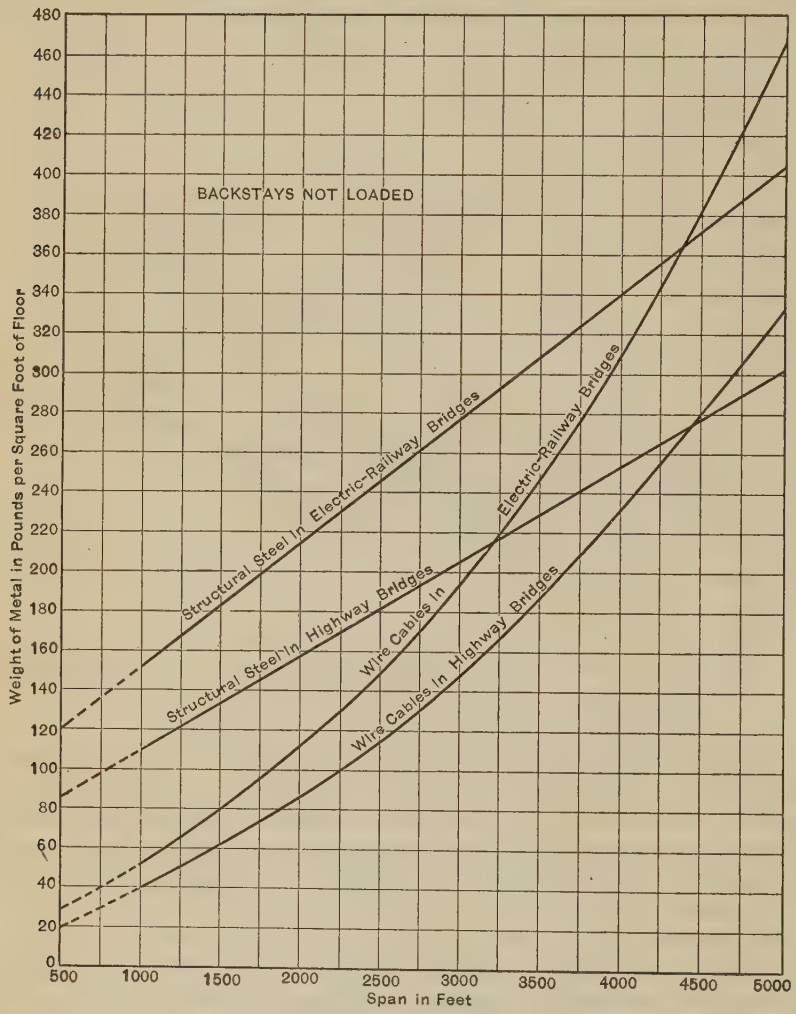

FIG. 16.—TOTAL WEIGHT OF METAL IN HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

The weights of structural steel and quantities of concrete in the pedestals are indicated in Figs. 18, 19, and 20. Fig. 18 is primarily for highway trestles. The classes *A*, *B*, and *C* adopted therein are those given for highway structures in the writer's standard specifications.*

* See "Bridge Engineering."

In case the structure is to carry electric-railway tracks, it will be necessary to add $(11 + 0.12h)$ lb. per sq. ft. of floor for the portion covered by the tracks, allowing a 10-ft. width for each track. If cantilevered footwalks are added, the weight of metal, w, per square foot for the additional area can be found by the formula, $w = 20 + 0.15h$, in which, h is the height of the trestle, in feet.

In case foundation piles are required for the pedestals, the number needed will be one for about every 80 sq. ft. of deck, irrespective of the length of the pile. This estimate is based on the use of small piles, each carrying 15 tons. These piles should generally be of reinforced concrete (about 12 in. square, or sometimes more). If wooden piles are used, they must be thoroughly protected from the atmosphere, and it will be necessary to increase somewhat the heights of the pedestals. For any change in the heights shown on Figs. 18 and 19 the quantities of concrete will vary almost proportionately.

The lighting of the approaches will cost, as in the case of the bridge proper, about \$0.10 or \$0.15 per sq. ft. of floor.

Miscellaneous Costs.—Some items of cost for a complete estimate cannot be included on a "per square foot of floor" basis, such as the retaining walls at the low ends of the trestle approaches, the earth embankments with their paving, the plazas for the fare-collecting booths of toll structures, right of way, and property damages. These items will have to be allowed for, at least approximately, in any estimate.

The cost of a retaining wall usually will lie between \$100 and \$125 per lin. ft.; and its equalized length can be taken as the full width of roadway and sidewalks plus 50 ft.

The cost of the plazas will depend greatly on the location, the amount of travel anticipated, and the architectural treatment. It can be stated for a one-way-travel plaza by the equation, $C = A\ w$, in which C is the total cost, in dollars, w is the width of roadway, in feet, for travel in one direction, and A is a coefficient varying from 2 000 for a country district to 3 000 for a city district. This assumes reasonably economic architectural treatment and the absence of retaining walls to sustain the fill. Under adverse conditions, this coefficient might have to be 2 000 larger to cover retaining walls and, in addition, 3 000, 4 000, or even 5 000 larger to cover elaborate or extravagant architectural treatment.

In respect to right-of-way costs and property damages, no estimate can be made, because they will depend entirely on the character of the locality through which the approaches pass. In a large city they will run high—sometimes into millions of dollars—but in country districts they may be small or even zero, if the property adjacent to the structure is benefited by the building of the bridge.

Applications

To illustrate the application of the foregoing methods, a specific case will be assumed and computations made for the completed structure, according to the following dimensions and ruling conditions:

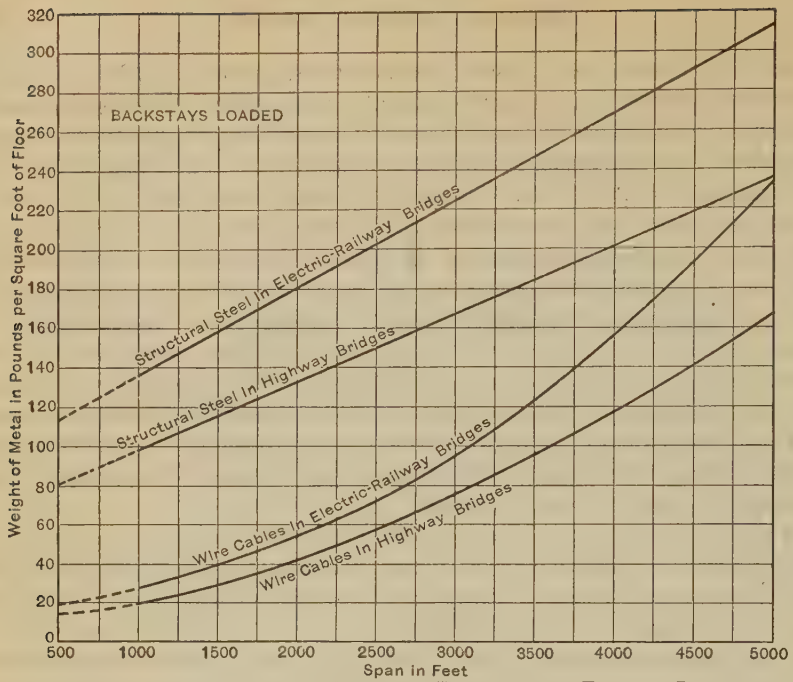

FIG. 17.—TOTAL WEIGHTS OF METAL IN HIGHWAY AND ELECTRIC-RAILWAY SUSPENSION BRIDGES.

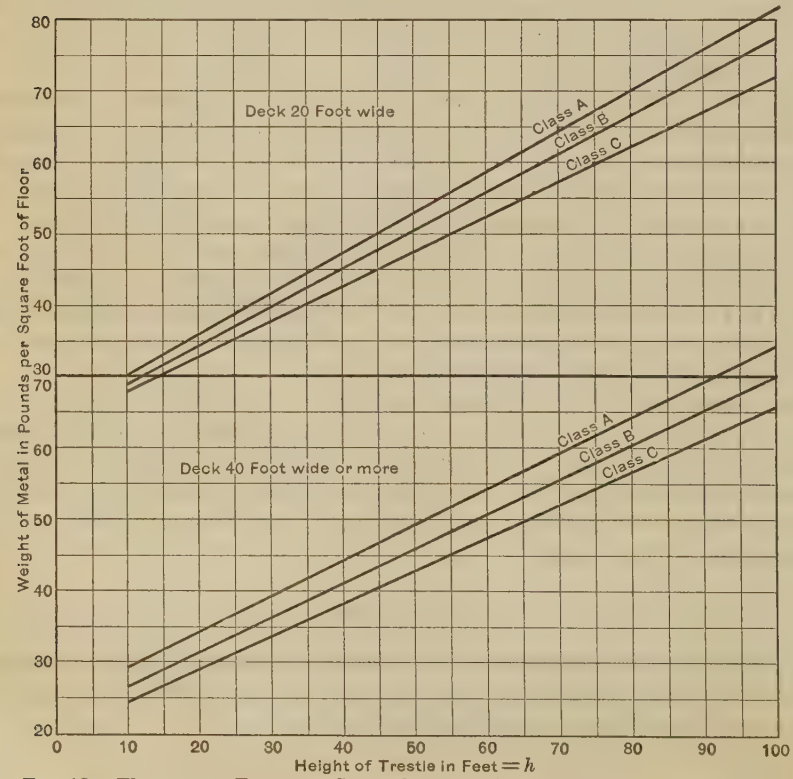

FIG. 18.—WEIGHT OF FLOOR IN STEEL TRESTLE APPROACHES TO HIGHWAY AND

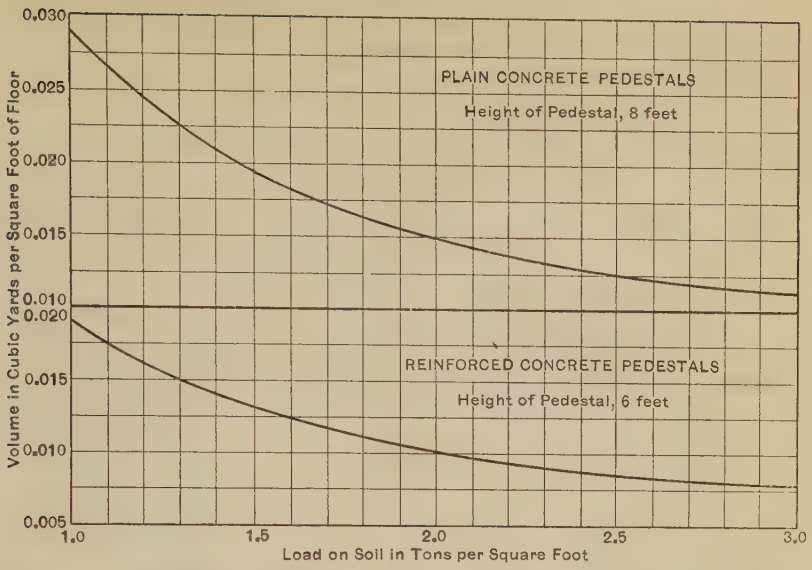

FIG. 19.—CONCRETE IN PEDESTALS WITH SPREAD FOUNDATIONS FOR STEEL TRESTLE APPROACHES TO HIGHWAY AND ELECTRIC-RAILWAY BRIDGES.

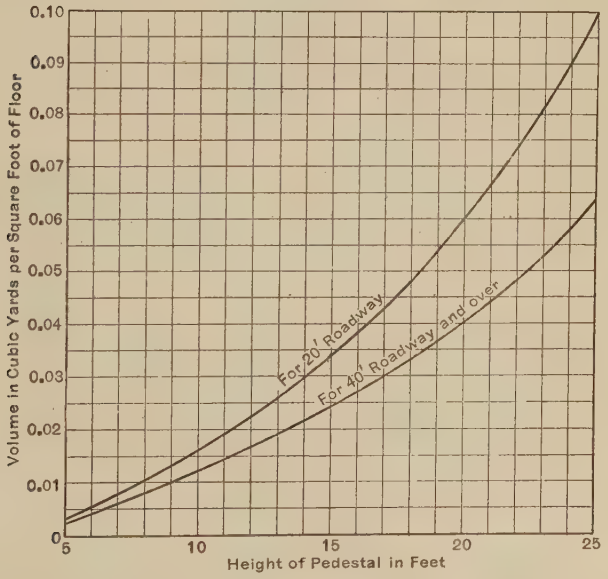

FIG. 20.—VOLUMES OF CONCRETE IN PEDESTALS ON ROCK FOUNDATIONS FOR STEEL TRESTLE APPROACHES TO HIGHWAY AND ELECTRIC-RAILWAY BRIDGES.

General

A toll structure, with a plaza on an embankment at the end of each approach for collecting tolls.
Tower encasement to be used.
Span and approaches to be elaborately lighted by electricity.
Estimated time to complete the bridge ready for traffic...6 years

Dimensions

Main span	3 800 ft.
No side spans, but trestle approaches up to the main span.	
Vertical Clearance above High Water:	
At middle of main span	150 ft.
At ends of main span	135 ft.
Variation between low and high water	10 ft.
Elevations at Foundation of Main Piers:	
One	0 ft.
One	—90 ft.
Height of Trestle (Ground to Grade):	
Near river bank	110 ft.
At outer end	20 ft.
Height of retaining wall (base to grade)	25 ft.
Total length of two approaches	2 080 ft.
Width of roadway in the clear, containing four lines of electric-railway tracks with open floor, located at mid-width	100 ft.
Width of sidewalk in the clear, each side cantilevered beyond the trusses	10 ft.

Financial

Cost of right of way and property damages.....$ 3 550 000
Interest on money required during construction..... 5 per cent.
Bonds to be sold at par, net.

Unit Prices.—

Silicon steel, in place	$0.085	per lb.
Nickel steel, in place	0.095	" "
Carbon steel in approaches, in place	0.060	" "
Wire cables, in place	0.16	" "
Pier shafts	22.00	per cu. yd.
Pier bases	38.00	" " "
Anchorages (on bare, dry rock)	15.00	" " "
Pedestals (on bare, dry rock)	20.00	" " "
Cost of base for pavement, including reinforcing metal	33.00	" " "
Cost of pavement	2.00	per sq. yd.
Cost of sidewalks, including reinforcing metal	33.00	per cu. yd.

Main Span

The cost of the main span is given in Table 2.

Approaches

Flooring.—The flooring will cost as follows:

Roadway, 60 sq. ft., @ $1.00	$60.00
Sidewalks, 20 sq. ft., @ $0.50	10.00
Railway, 40 sq. ft., @ $0.70	28.00
Total cost of flooring per linear foot of structure	$98.00

TABLE 2.—Cost of Main Span.

Item.	Quantity per square foot of roadway.	Unit price.	Product.	Equivalent area, in square feet per linear foot.	Cost per linear foot of span.
Highway slab	0.02 cu. yd.	$33.00	$0.66	60	$39.60
Pavement	0.11 sq. yd.	2.00	0.22	60	13.20
Sidewalk slabs	0.012 cu. yd.	33.00	0.40	20	8.00
Railway floor	.....	.....	0.70	40	28.00
Floor system, highway (Fig. 3)	50 lb.	0.095	4.75	70	332.50
Floor system, railway (Fig. 3)	52 lb.	0.095	4.94	40	197.60
Lateral system, average of highway and railway (Fig. 4)	16 lb.	0.095	1.52	110	167.20
Stiffening trusses, highway (Fig. 5)	67 lb.	0.095	6.37	70	445.90
Stiffening trusses, railway (Fig. 5)	102 lb.	0.095	9.69	40	387.60
Suspenders, average of highway and railway (Fig. 7)	7.5 lb.	0.095	0.71	110	78.10
Main cables, highway (Fig. 8)	116 lb.	0.16	18.56	70	1 299.20
Main cables, railway (Fig. 8)	156 lb.	0.16	24.96	40	998.40
Back-stays, highway (Fig. 9)	94 lb.	0.16	15.04	70	1 052.80
Back-stays, railway (Fig. 9)	124 lb.	0.16	19.84	40	793.60
Towers, highway (Fig. 10)	85 lb.	0.085	7.22	70	505.40
Towers, railway (Fig. 10)	114 lb.	0.085	9.69	40	387.60
Tower encasement (Fig. 13)	.....	.....	4.10	110	451.00
Lighting	.....	.....	0.15	120	18.00
Pier shafts, highway (Fig. 11)	0.141 cu. yd.	22.60	3.10	70	217.00
Pier shafts, railway (Fig. 11)	0.152 cu. yd.	22.00	3.34	40	133.60
Pier bases, highway (Fig. 12)	0.069 cu. yd.	38.00	2.62	70	183.40
Pier bases, railway (Fig. 12)	0.073 cu. yd.	38.00	2.77	40	110.80
Anchorages, highway (Fig. 14)	.....	.....	5.70	70	399.00
Anchorages, railway (Fig. 14)	.....	.....	8.20	40	328.00
Total cost per linear foot of bridge proper	.....	.....	.....	.....	$8 375.50

Lighting.—

Cost per linear foot of structure $18.00

Trestle 110 Ft. High.—

Unit Quantities:

Metal (Class A), (Fig. 18), for roadway ..... 80 lb.
Metal (Classes 25 and 30), (Fig. 18), for railway .... 104 lb.
Metal (Class C), (Fig. 18), for sidewalks ..... 36 lb.
Metal hand-rails, 4 @ 70 lb. per lin. ft. ..... 280 lb.
Pedestals, 9 ft. high on rock (Fig. 20), 0.01 cu. yd. per sq. ft. of floor.

Quantities per Linear Foot:

Roadway, 60 sq. ft. @ 80 lb. ..... = 4 800 lb. per lin. ft.
Railway, 40 sq. ft. @ 104 lb. ..... = 4 160 " " " "
Sidewalks, 20 sq. ft. @ 36 lb. ..... = 720 " " " "
Hand-rails, 4 lines @ 70 lb. ..... = 280 " " " "

Total weight of metal ..... 9 960 lb. per lin. ft.
Say ..... 10 000 lb. per lin. ft.

Pedestals, 0.01 × 120 = 1.2 cu. yd. per lin. ft.

Summary of Cost per Linear Foot of Trestle:

Metal, 10 000 lb. @ $0.06 ..... $600.00
Flooring 98.00
Lighting 18.00
Pedestals, 1.2 cu. yd. @ $20.00 ..... 24.00

Total cost per linear foot of trestle 110 ft. high $740.00

Trestle 20 Ft. High.—

Unit Quantities:

Metal (Class A), (Fig. 18), for roadway.... 33 lb.
Metal (Classes 25 and 30), (Fig. 18), for railway..... 47 lb.
Metal (Class C), (Fig. 18), for sidewalks.. 23 lb.
Metal hand-rails, 4 @ 70 lb.....280 lb. per lin. ft.
Pedestals, 5 ft. high, on rock (Fig. 20), 0.002 cu. yd. per sq. ft. of floor.

Quantities per Linear Foot:

Roadway, 60 sq. ft. @ 33 lb.....	=	1 980 lb. per lin. ft.
Railway, 40 sq. ft. @ 47 lb.....	=	1 880 " " " "
Sidewalks, 20 sq. ft. @ 23 lb.....	=	460 " " " "
Hand-rails, 4 lines @ 70.....	=	280 " " " "
Total weight of metal.....		4 600 lb. per lin. ft.

Pedestals, $0.002 \times 120 = 0.24$ cu. yd. per lin. ft.

Summary of Cost per Linear Foot of Trestle:

Metal, 4 600 lb. @ $0.06.....	$276.00
Flooring	98.00
Lighting	18.00
Pedestals, 0.24 cu. yd. @ $20.....	4.80
Total cost per linear foot of trestle 20 ft. high.....	$396.80
Say	$400.00

Average Cost of Trestle.—

Cost per linear foot for 110 ft. height.....	$740.00
" " " " " 20 ft. "	400.00
	$1 140.00
Average cost per linear foot for trestle.....	$ 570.00

Retaining Walls.—

Length, $2 \times (120 + 50)$.....	340 ft.
Cost per linear foot.....	$ 100
Total cost of retaining walls.....	$34 000

Grand Total Cost of Project

Main structure, 3 800 ft. @ $8 375.....	$31 825 000
Approaches, 2 080 ft. @ $570.....	1 186 000
Retaining walls	34 000
Two plazas, $30 \times 3\,000$.....	180 000
Right of way and property damages.....	3 550 000
Summation	$36 775 000
Contingencies at 5%.....	1 838 000
Summation	$38 613 000
Engineering and administration at 8%.....	3 089 000
Summation	$41 702 000
Interest during construction at 15%.....	6 255 000
Grand total cost of project.....	$47 957 000
Say	$48 000 000

Deductions

In case there are side spans with decks and stiffening trusses suspended from the cables, the procedure in making an estimate is to ascertain the total cost per linear foot for the superstructure of the main span between towers, omitting the item of back-stays, and apply it to the combined lengths of the main span and side spans. The total costs of the towers, piers, and anchorages are about the same whether the back-stays carry vertical loads or not.

It is true that the stiffening trusses for the side spans are slightly heavier than those for the main span, usually 4 or 5%; but this increase is partly offset by a saving in the weight of the lateral system, because the span is only one-half as long as the main span, and therefore the diagonals are lighter. The longer the spans, the greater will be the proportionate saving from this cause. On the whole, the side spans will be a little more expensive per square foot of floor than the main span. As the cost of the stiffening trusses is generally only about 10% of the total for the superstructure, it is evident that the error caused by assuming equal unit costs for the side spans and the main span is negligible. However, in preparing Fig. 17 cognizance was taken of this slight variation.

These studies aid in determining the extreme feasible lengths of suspension spans. The criterion of feasibility, however, is a mooted point. The writer in his investigations concerning the use of alloy steels in bridgework* assumed for cantilever structures that the limit of feasibility is reached when it requires 4 lb. extra of metal to support 1 lb. additional live loading. The best possible criterion for a suspension bridge, however, is the financial ability of the interested communities to raise the necessary funds for its construction.

Referring to Fig. 15, it is evident that this limit has not been exceeded even for electric-railway structures with a span of 5 000 ft.; for, with anchorages on pile foundations, the cost per square foot of floor is only $165. As the minimum clear width of roadway for this span is 130 ft., the cost of the bridge proper would be $130 \times 5\,000 \times 165 = \$107\,250\,000$; but to this must be added a large amount for approaches, right of way, property damages, contingencies, engineering, administration, and interest during construction. Judging from the preceding example, the grand total cost would amount to about $150 000 000. Under certain conditions, this amount is not beyond the limit of feasibility, although it is probably outside that of economics for a toll structure, considered as a business venture. For example, if it should require that amount to build a 5 000-ft. span across The Narrows of New York Harbor, and if the adjacent communities in New York and New Jersey were to demand such a structure, the money undoubtedly could be raised.

Computations of the "approximate greatest lengths of span" have been made† by William H. Burr, M. Am. Soc. C. E., based on certain assumed unit stresses for cables and various ratios of sag to span. His tabulated results are such that, if any additional loading were placed on the structure, the

* "Nickel Steel for Bridges," *Transactions*, Am. Soc. C. E., Vol. LXIII (1909), p. 101, and "The Possibilities in Bridge Construction by the Use of High-Alloy Steels," *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 1.

† "Suspension Bridges," p. 74.

cables would be over-stressed, no matter how much area were added to their cross-section. Using the writer's assumptions, Mr. Burr's tables would indicate an extreme limit of span of about 10 000 ft. This limit satisfies one's curiosity, but does not give information concerning the true or practically feasible limiting span. In the writer's opinion, that limit is reached at about 5 000 ft.; but, in order to show what would occur beyond it, Fig. 21 has been

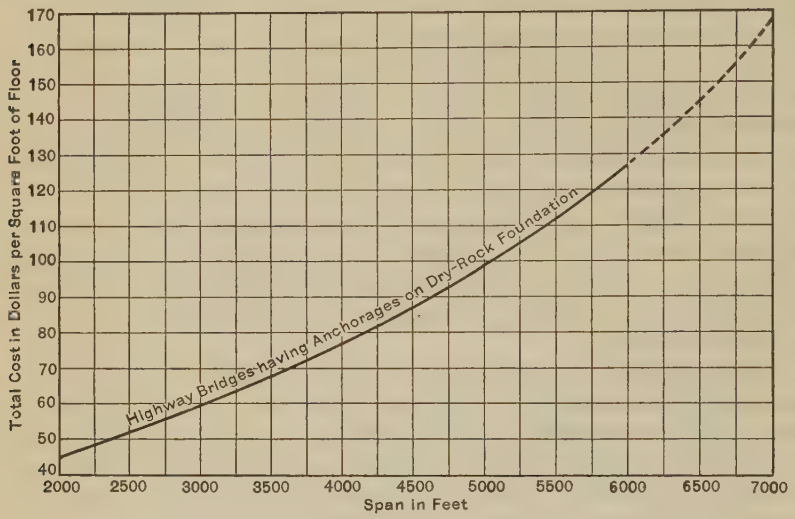

FIG. 21.—TOTAL COST OF LONG-SPAN HIGHWAY SUSPENSION BRIDGES.

prepared by actually computing the quantities and the costs for a 6 000-ft. span highway bridge, and by extrapolation (as shown by the dotted line) up to a span length of 7 000 ft. For this illustration the lowest line of Fig. 15, representing highway bridges with anchorages resting on bare rock, was chosen as being the most likely condition. From it is made the following estimate of total cost of project for a highway suspension bridge of 7 000-ft. span:

Bridge proper, 1 260 000 sq. ft. @ $166	$209 160 000
Approaches, say, 2 600 ft. @ $600	1 560 000
Retaining walls, say	40 000
Two plazas, say	300 000
Right of way and property damages, say	3 800 000
Summation	$214 860 000
Contingencies, 5%	10 743 000
Summation	$225 603 000
Engineering and administration, 8%	18 048 000
Summation	$243 651 000
Interest during construction $\frac{1}{2} \times 5\%$ for 8 years = 20%	48 730 000
Summation	$292 381 000
Grand total cost of project, say	$300 000 000

This, of course, is a far greater amount than any conditions at present conceivable would warrant.

A similar calculation for a 6 000-ft. span makes the grand total cost of project nearly $180 000 000, which probably is also too great for practical consideration.

Figs. 16 to 20, inclusive, make possible a comparison of the costs of suspension bridges with and without suspended side spans. Compare the case already computed, namely, a 3 800-ft. span, assuming 1 900-ft. approaches, with a similar bridge that has suspended side spans of 1 900 ft. each. The deck, the main piers, the anchorages, the tower encasement, and the lighting will cost nearly the same in both cases, and hence may be ignored.

Cost of Metal

One-Span Structure.—

Structural steel, highway (Fig. 16)	240 lb. @ $0.094 (ave.)	× 70 ft. =	$1 579.20	per lin. ft.
Cables, highway (Fig. 16)	216 " @ 0.16	× 70 ft. =	2 419.20	" " "
Structural steel, railway (Fig. 16)	330 " @ 0.094 (ave.)	× 40 ft. =	1 240.80	" " "
Cables, railway (Fig. 16)	284 " @ 0.16	× 40 ft. =	1 817.60	" " "
Total			$7 056.80	per lin. ft.

Total cost of metal = 3 800 × $7 056.80 = $26 816 000

Trestle Approaches for One-Span Structure.—

Metal	10 000 lb. @ $0.06	= $600	per lin. ft.
Pedestals	1.2 cu. yd. @ $20	= 24	" " "
Total		= $624	per lin. ft.

Total cost of trestle = 3 800 × $624 = $2 371 000

Three-Span Structure.—

Structural steel, highway (Fig. 17)	190 lb. @ $0.094 (ave.)	× 70 ft. =	$1 250.00	per lin. ft.
Cables, highway (Fig. 17)	110 " @ 0.16	× 70 ft. =	1 232.00	" " "
Structural steel, railway (Fig. 17)	258 " @ 0.094 (ave.)	× 40 ft. =	970.10	" " "
Cables, railway (Fig. 17)	140 " @ 0.16	× 40 ft. =	896.00	" " "
Total			$4 348.30	per lin. ft.

Total cost of metal = 7 600 × $4 348.30 = $33 047 000

Comparison of Costs

Cost of one-span bridge	$26 816 000
Cost of approaches	2 371 000
Comparing cost for one-span structure	$29 187 000
" " " three-span "	33 047 000
Difference in favor of one-span structure	$-3 860 000

The total cost of the one-span structure would be:

Main structure, 3 800 ft. @ $8 375	$31 825 000
Approaches, 3 800 ft. @ $740	2 812 000
Retaining walls	34 000
Total comparing cost	$34 671 000

Percentage of increase for three-span structure:

$$3\,860\,000 \times 100 \div 34\,671\,000 = 11, \text{ nearly.}$$

This is good evidence of the correctness of a claim made by the writer many years ago, namely, that there is no economy in adopting suspended side spans for suspension bridges when steel trestles or even short, independent spans can be used instead.

Conclusion

The computations for this paper have been carefully checked, hence the results are reliable. Nevertheless, owing to the meager treatment to date of the design of long-span suspension bridges, it is probable that another engineer in making a similar investigation would obtain somewhat different results, even if he were to adhere to the same specifications for design.

DISCUSSION

T. Kennard Thomson,* M. Am. Soc. C. E.—It is to be hoped that Mr. Waddell will omit the statement that the data he gives would enable one to estimate the cost of a suspension bridge in a few hours. Only a very experienced engineer could do that and he only after spending weeks in studying the site and local conditions. Such an engineer would probably object to accepting this information second-hand. The statement might encourage many a business man to think that he could make the estimate himself—or hire an inexperienced engineer to do so.

The speaker once met a president of a bridge company who was not an engineer, but who had a copy of the Carnegie Steel Company's "Pocket Companion" and had "designed" a number of plate girders by using the bending moment at the center of the span and assuming that half way between the end and the center the moment would be exactly one-half the maximum. From this he reasoned that he could use two angles having the same cross-sectional area as the cover-plate—allowing him to make the length of the cover-plate exactly one-half the span.

Thousands of other cases could be cited (some involving internationally known disasters) to show the danger of putting tools or formulas in the hands of a novice or inexperienced engineer.

As for accepting information about the site, etc., without knowing the personal equation of the investigator—the speaker once received a letter describing the local condition of what he thought was a proposed building, with a request to wire what his charges would be for a report on it. He was on the point of doing so when something made him pause, and then wire that he would make no report without seeing the site first. No answer was received to that, but, later, he heard that the building had been constructed and had collapsed before he was asked to report. In other words, the architect, more than 500 miles away, was trying to get the endorsement of his plans by a distant engineer.

Why should the author give even an approximate formula for the cost of the foundations? Nothing in the cost of bridgework varies more than this. Many other items besides the depth affect it. For instance, the water might be a calm lake or bay, or a raging torrent; or both. While constructing a bridge across the Ohio River, part of the time the depth was 6 ft. with practically no current, and during the same year it was 106 ft.—a violent flood.

The author gives some reasons against the use of eye-bar suspension bridges. Many such bridges have been built and are still standing—but one point which should carry considerable weight when considering long-span suspension bridges is that, while the assumption would be that the pins would be as free to move as an axle, they are, obviously, never so designed. The result is that the pins sooner or later become almost as fixed as a riveted joint, or the pin-holes become slotted. As a proof of the first statement, tests have

* Cons. Engr., New York, N. Y.

been made of the stresses in the eye-bars of suspension bridges, with the result that the testing machine indicated compression on the top of the eye-bar and tension on the bottom of the same bar—showing conclusively that the pins had become bound.

Mr. Waddell suggests a concrete casing not in contact with the steel. This is just the reverse of the speaker's experience, for he has found that where good concrete was in direct contact with the steel, no rusting had started; but that where there was a space between the steel and the concrete, the metal was badly pitted.

To allow only 5% for contingencies, even after the most exhaustive studies, seems absolutely inadequate, for many reasons, such as labor conditions, accidents, national finances, or individual and personal equations. For instance, a very able man, experienced in his line,* took a Government contract, estimating the exact amount of money he would require, and made his banking arrangements accordingly. Through no fault of his, payments were temporarily withheld and he had to go back to his bankers who then "wiped him out."

In making these few suggestions, the speaker well knows that it would take pages to point out the many valuable features of this admirable paper.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—As a large part of the total cost of any highway bridge is a direct effect of the weight of the floor itself with its pavement, if any, it would appear that there is here a large field open to invention in devising some type of light weight floor that will not sacrifice the many advantages which the reinforced concrete slab possesses. Assuming a certain strength as necessary in any event, the main requirements are immunity against rot and the risk of damage by fire. The light-weight aggregate mentioned by the author is not unlike pumice stone in texture and would seem to the writer to be open to the objection of extreme porosity. This would invite the penetration of moisture to the reinforcing steel with the consequent danger of rust and ultimate failure of the slab.

Timber floors have been very generally and properly condemned in the past for important highway bridges and their use has been discouraged except in cases where the cheapest possible construction has been imperative. It may be, however, that the fault has been in the way in which the material has been used rather than in the material itself. The objection to timber in bridge floors is its tendency to rot and also the possibility, which in many cases is a probability, of damage by fire that may extend to the bridge itself if not caught in time.

In the great majority of cases decay in timber is the result of attack by fungi. There are other causes but they are seldom operative in open-air construction. Decay may start on any surface where the spores can find lodgment, or it may come from infection prior to the time the material is put into place. Some sort of antiseptic treatment is about the only means of guarding against it.

* Cons. Engr., Vancouver, B. C., Canada.

While fire may start on any exposed surface, it can not maintain itself unless the oxygen supply can be kept up. Conditions are favorable to continued combustion on the under side of horizontal slabs or in crevices that extend through the mass. They are unfavorable on the upper side or in blind crevices where air circulation is impossible. The ordinary type of timber floor construction has been such as to invite decay and to encourage combustion. The planks have been laid flat in one or more layers with plenty of air spaces between them so as to offer exceedingly favorable breeding grounds for fungi and the most favorable conditions possible for propagation of fire.

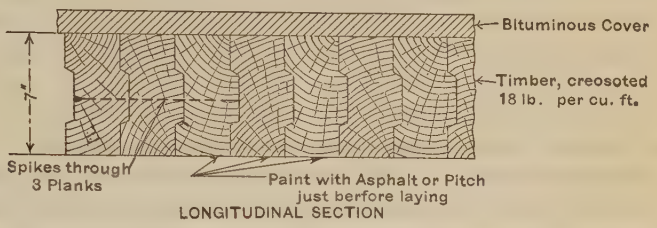

FIG. 22.—TIMBER FLOOR CONSTRUCTION.

The type of timber floor construction illustrated in Fig. 22 was suggested to Mr. Waddell some time ago by the writer in connection with a particular bridge problem where cost was an important factor in the decision as to whether or not to build a bridge. The floor planks should be sawn and dressed before seasoning to avoid checks as far as possible. They should be kiln-dried and then saturated with creosote, or other antiseptic, to the limit of their capacity of absorption under the vacuum and pressure process. Before laying, the faces in contact should be blotted dry and painted with a film of asphalt or a similar material, and laid with the paint still soft. A wearing surface of bituminous concrete will serve as a protection from dirt and the ever-present cigarette and cigar butt. A floor of this construction would weigh about 50 lb. per sq. ft. as against 98 lb. for ordinary concrete or 82 lb. for the light-weight aggregate.

The main requirement in the anchorage, apart from architectural considerations, is weight and, in many cases, this might be provided by a cheaper material than concrete. The writer offers the suggestion that the anchorages be made in the form of a reinforced concrete box filled with sand, or sand and gravel, which could be pumped into place at small expense. The boxes could be built in much the same way as elevator bins by the use of moving forms.

In respect to the tower encasements which the author has included, the writer fails to see the necessity of ever going to this expense. Their only purpose could be to satisfy the requirements of esthetics and although there may be a lingering idea that stone towers are the only proper thing for a suspension bridge, the writer believes this is more a result of tradition than of any real improvement in appearance. Properly treated, steel towers can be made quite as effective, possibly more so, than masonry. The contrast

between the Brooklyn Bridge and the Manhattan Bridge is not at all to the disadvantage of the latter; and for a horrible example of the effects of disguising the truth there is the Tower Bridge in London. The towers of that bridge were supposed to be designed to harmonize with the massive construction of the ancient Tower of London close by, but it cannot be said that the desire has been realized; at least, not in the opinion of the architects of those parts if one can trust the published remarks of one of them. Mr. Heathcote Statham, a British architect, condemns it as,

"An example of a different kind; it represents the vice of tawdriness and pretentiousness, and of falsification of the actual facts of the structure. * * * All architects would have preferred the plain steel structure to this kind of sham. If iron is designed on good lines it will look better in itself without these gewgaws".

Allston Dana,* M. Am. Soc. C. E.—The method used by the author for determining the weight of stiffening trusses has already been discussed. As the speaker understands it, the trusses were designed for convenience by the so-called elastic theory (which has been known for some time to give unnecessarily heavy trusses), and the weights were then scaled down by certain ratios so as to obtain weights corresponding with trusses designed by the so-called deflection theory (which has been recognized as giving correct results). The ratio used by the author was that found in the case of the stiffening trusses used in the Delaware River Bridge. These trusses were designed by the deflection theory, and the comparison with trusses designed by the erroneous elastic theory was made later only as a matter of interest and to bring out the "saving" resulting from using a correct analysis.

This correct analysis is not really based on any particular theory but rather on the actual conditions which must obtain. These conditions are that under the influence of temperature and live load, full or partial, the cables change shape or deflect, and at all points the trusses must deflect the same amount. The distribution of the total load between the cables at each point must be such that their deflections are the same throughout. This distribution may be found by a cut-and-try process or by more formal methods. If there were no trusses the cables would deflect so as to conform to the force polygon of the loads. This natural deflection is dampened or resisted by the use of stiffening trusses to a greater or less extent, depending on the relative stiffening effect of the trusses.

Two extreme cases may be considered. First is that in which the trusses are relatively so stiff as compared with the cables that practically they resist entirely the deflection of the cables. This is likely to be the case for very short spans, where the large ratio of live load to dead load would make very stiff trusses desirable. For this case the elastic theory would be practically correct, because this theory assumes that there is no deflection of the cables or, in other words, that partial live loads are uniformly distributed by the trusses to the cables.

The second extreme case is that in which the trusses are relatively so flexible that they offer practically no resistance to the deflection of the cables.

* Engr. of Design, The Port of New York Authority, New York, N. Y.

This is likely to be the case for very long spans where the small ratio of live load to dead load would make stiff trusses unnecessary. For this case the design of the trusses would consist merely in finding the unit stresses in the chords corresponding to the curvature into which the trusses would be forced by the deflection of the cables. The unit stress for any given amount of curvature is directly proportional to the truss depth, and obviously a depth would have to be selected which would give permissible unit stresses. The determination of the chord areas in such a case would be a matter of selecting a nominal cross-section, of sufficient size to justify the compressive unit stress.

These two extreme cases consist then either in assuming absolutely stiff trusses and designing them for given loads, using a depth to give an economical design, as in simple truss bridges; or in assuming absolutely flexible trusses and designing nominal sections, with a truss depth and unit stresses that correspond. Of course, no trusses are either absolutely stiff or absolutely flexible, but for very short spans the first assumption would be approximately correct, and for very long spans the second assumption would be approximately correct. In this latter case, the weight of trusses would be many times lighter than those assumed by the author. For what may be called intermediate lengths, however, neither of these methods would be satisfactory. The Delaware River Bridge is a good example of an intermediate case because the trusses as designed reduce the cable deflections to about one-half the natural or unstiffened amounts. To have designed the trusses by the first method would have given a result not even approximately correct. To have designed them by the second method would have necessitated a depth of truss about one-half that actually used, in order to keep the unit stresses down to the permissible amounts. Such shallow nominal trusses were not considered adequate, however, as it was felt desirable to resist to a considerable extent the motion of the cables and towers. There remained then no escape from the somewhat more complicated method of correctly analyzing the deflections of the combined cable and truss system, and finding the load distribution and the bending moments in the trusses. This may be looked at as a process of selecting both the truss depth and the allowable unit stresses, and then of computing the necessary chord areas to give trusses of such stiffness that the cable deflections will be dampened sufficiently to keep the resulting truss curvature down to that corresponding with the depth and allowable unit stresses. If somewhat shallower trusses had been used their weight would have been much less, or if somewhat deeper trusses had been used their weight would have been much greater, so that the particular ratio found between the weight of those used and that of trusses designed by the elastic theory is a function of the depth.

For the author to take the "ratio" found for the Delaware River Bridge for the purpose of finding truss weights for all lengths of spans is, therefore, equivalent, not only to agreeing with the depth used for that length of span, but of tacitly assuming certain depths for all other span lengths, depths which there is no reason to believe are the same as those assumed in

Table 1 of the paper. The very pertinent criticism that the truss weights as given in Fig. 5 are much too great for the longer spans is therefore equivalent to stating that the truss depths as given in Table 1, or as indirectly established by the assumed "ratio", are unnecessarily large for the longer spans. With this the speaker is in agreement, it being his opinion that a truss depth of about 30 ft. would give a very satisfactory truss for all length of spans.

With regard to the use of over-head bracing the author states that in all the designs "he has used over-head bracing, as, on general principles, he is opposed to pony trusses, especially in long spans". He further states:

"It is true that the tension in the suspenders tends to keep the top chords of pony trusses from getting greatly out of line; but when computing the top chords, it would be difficult to determine satisfactorily the proper value of the slenderness ratio".

In suspension bridges with only two trusses the trusses are naturally quite far apart and over-head bracing if used would be very heavy and would also add to the cost of the cables and towers. It seems unfortunate, therefore, that the author has dismissed so summarily the matter of omission of over-head bracing and for the reasons given.

The top chords of pony trusses are held in line not by the tension of the suspenders but by the web members of the trusses, especially the verticals; and the adequacy of such support depends on the depth of the trusses and not on the length of the span. The length of span hardly enters into the question, for, as mentioned previously, the truss depth is or should be independent of the span length. In the case of the Delaware River Bridge it was decided to omit the over-head bracing both for economy and for appearances. The speaker made a thorough investigation of the stresses in the top chord and verticals due to the omission of the bracing. The verticals not only hold the top chord in line but also tend to force the chord out of line due to the deflection of the floor-beams from live load. With floor-beams fully loaded between trusses and the cantilever projections empty, the maximum inward deflection of the verticals occurs. With the cantilevers fully loaded and with no load between trusses, the maximum outward deflection of the verticals occurs. Assuming alternate loading condition of this kind along the bridge, the top chord would be forced into a series of lateral waves. The shortest distance between points of contraflexure is two panels. For this case it was found that the lateral stiffness of the chord was sufficient to resist almost completely the deflection of the verticals, forcing them to bend back. The lateral force caused by the change in direction of the chord under compression was of course included in the calculations.

Where the points of contraflexure were far apart, however, the chord was found to be insufficiently stiff to resist deflections and the lateral thrust from the change in direction of the chord deflected the verticals somewhat more, being resisted by the stiffness of the verticals and floor-beams. Cases were investigated for various distances between points of contraflexure and the bending stresses in chords and verticals computed. In no case was the

chord bending stress found to be greater than 3 000 lb. per sq. in., and the allowable compressive unit stress was reduced by that amount for designing the chord members. The value of the slenderness ratio mentioned by the author as being uncertain for pony trusses, thus did not have to be determined.

I. Oesterblom,* M. Am. Soc. C. E. (by letter).—Having had occasion to inquire into the matter of light-weight concrete, the writer's attention was called to "Haydite", to which Mr. Waddell makes reference, and regarding which significant claims are made, evidently well substantiated by both experience and tests. At the time of the inquiry copies of several test certificates were given, a few extracts from which are as follows:

TABLE 3.—Tests on "Haydite" Concrete.

Reference.*	Test No.	Mixture.	Wetness, percentage.	Weight, in pounds per cubic foot.	Compressive stress, in pounds per square inch.	Tensile stress, in pounds per square inch.	Age of sample, in days.	Notes.
A..	H H 253	1 : 3 : 5	3.5	93	1 290	.....	28	.....
B..	H H 253	1 : 3½ : 5	3.5	101	1 621	.....	28	.....
C..	30 846	1 : 2 : 2	21.6	1 048	3 620	.....	28	.....
D..	30 846	1 : 1 : 2	20.4	106	4 350	.....	28	.....
E..	.....	1 : 1¼ : 1¼	.....	112	3 360	.....	.....	*E* = 1 940 000 lb. per sq. in.
F..	1920-37	1 : 3	.....	.....	.....	418	31	.....
G..	1920-37	1 : 3	.....	.....	.....	567	90	.....
H..	1920-27	1 : 2 : 2	.....	.....	3 308	.....	28	.....
I...	1920-27	1 : 2 : 2	.....	.....	3 919	.....	90	.....
J...	1920-26	1 : 2 : 4	.....	.....	2 065	.....	28	.....
K..	78 409-79 061	1 : 2 : 4	"Normal"	107	3 268	.....	30	.....
L..	82 028	1 : 2 : 4	"Plastic"	115	3 082	.....	30	.....
M..	82 309	1 : 1 : 7:2.	"Plastic"	118	4 523	.....	30	.....
N..	87 831	1 : 2 : 4	"Field"	116	2 520	.....	28	.....
O..	88 571	1 : 2½ : 3	.....	97	2 738	.....	28	110½ lb. per cu. ft., wet.

* *A* and *B* = Kansas City Testing Laboratories, Inc.; *C* and *D* = Bureau of Standards, Washington, D. C.; *E* = Lewis Inst., Chicago, Ill.; *F* to *J* = Municipal Laboratory, Kansas City, Mo.; *K* to *O* = Kansas City Testing Laboratories, Inc. On control tests for actual building operations, as follows (all in Kansas City); *K* = Deaner Dental Clinic Bldg.; *L* = Westinghouse Bldg.; *M* = Kansas City Star Bldg.; *N* = Seested Bldg.; *O* = Central Junior High School. The test reports often had the remark that the samples were weighed wet.

Table 3 gives extracted averages in regard to weight and strength, also information about mixture, consistency, and the ages of the test pieces. Reference is also made to the nature of the tests, many of them having been made as control tests on actual building operations; the names of the laboratories are also given.

From Table 3 it will be seen that the compressive strength is considerable. Evidently, the "Haydite" aggregate itself is equal or superior to the enclosing cement mortar. The same is true, perhaps in a larger measure, also in regard to the tensile strength. The modulus of elasticity is quite equal to normal.

* Engr., Bldg. Dept., Faitoute Iron & Steel Co., Newark, N. J.

There are additional qualities, shown in the certificates, but not given in Table 3, which are also noteworthy. "Haydite" concrete is most thoroughly water-proof, which ordinary concrete even when well made is not; it is also a much safer material in very intense fire.

Most of the commonly used aggregates have been produced by the colloidal process; they are, therefore, full of capillaries and very water-absorbent. The cement mortar is similar in nature and likewise water-absorbent. The "Haydite" aggregate is produced by a burning process; it is full of bubble-like cavities, or small cells, but these are enclosed by walls, produced in fire and perfectly impervious. It is in the nature of a cellular material to aid in a more thorough hydration of the cement, thereby densifying the structure of the mortar. The two features combine to make "Haydite" concrete very impervious.

It is of less significance in bridge construction that "Haydite" concrete is especially resistant to intense fire. It is obvious that it should be so. Having been produced in intense fire there can be no further changes of a chemical nature when the material is again exposed to intense heat. "Haydite" is, therefore, proof to both elements—fire and water.

If the future should bear out what the past already seems to have well established, it is clear that "Haydite" concrete is the material for all bridge floors, and indeed for supporting elements as well in girder and arch bridges. The more the dead weight enters as an important element of construction, the more necessary will be the use of "Haydite" concrete or some similar material possessing equal advantages.

Leon S. Moisseiff,* M. Am. Soc. C. E.—This paper on the cost of long-span suspension bridges is of much interest to bridge engineers. The author has heretofore been the chief contributor to the studies on bridge economics published in the United States. This paper thus forms the logical continuation of his investigations.

Studies in bridge economics in English are rather scant and of such as have been made only a few have been published. Engineers of other countries have given much attention to the subject but their studies have been practically limited to short-span bridges of the simpler types. For this there was good reason. When Governments own the railroads and control by decrees the applied loads and the allowable materials and stresses and prescribe the standards of the designs to be followed, the study of the cheapest bridge within the law becomes much simplified—a matter of comparing types and spans based on exactly the same requirements and the same allowances. In fact, European studies are frequently referred to as based on the "Regulations of A. D. so and so". A change in the Governmental decrees is, therefore, soon followed by a new study of the economics it involves.

For the same reason European engineers, with a few purely academic exceptions, have not gone deeply into the study of the economics of long-span bridges. Another more important reason is that there are not enough long-span bridges on the European continent to necessitate such studies. The

* Cons. Engr., New York, N. Y.

same reason holds true in the United States for suspension bridges of the enormous spans investigated by the author. Therefore, the necessity of this paper and its use for suspension bridges of very long spans is not obvious. Suspension bridges of the kind studied are large and costly; they are not built every day. During the last twenty years, that is, since the construction of the Manhattan Bridge, few large suspension bridges have been built in the world. They can be counted on the fingers of one hand: The Bear Mountain and Delaware River Bridges, in the United States, the Florianopolis Bridge, in Brazil, and the Cologne Bridge, over the Rhine.

Long-span bridges are expensive either relatively because of the light available traffic or absolutely because dense traffic means wide roadways, heavy loads, and thickly populated cities with correspondingly expensive land. Projects requiring large funds take considerable time to ripen. The demand for them must be persistently felt before public or private agencies undertake their realization. Some well-known bridge projects have taken decades in developing. Engineers, therefore, will have more than enough time to study the proposed bridge and estimate its cost. They will hardly be compelled to read off the cost of the project from diagrams based on materials, fabrication, and estimates of cost, which may be established in the year of grace 1926.

All these remarks do not in the least lessen the intrinsic value of the paper. A great amount of work has been done by the author and when the occasion arises the planning engineer will make good use of it.

In his study of stiffening trusses the author refers to the speaker's paper* on the Camden Bridge, showing that "for the stiffening trusses of the main span, 1 750 ft. in length, the deflection theory reduces the weight of the metal to 64% theoretically, or 67% actually, of that found by the elastic theory, the difference in these percentages being due to the use of certain minimum sections in designing."

This is, therefore, an opportunity to make a short statement on the deflection theory. This theory differs from the elastic theory, as developed by Mueller-Breslau, Melan, and others, in that in the latter it is tacitly assumed that the cable curve remains undistorted after the advent of load on the bridge, while the former takes cognizance of the distortion of the structure. Thus it follows that in the elastic theory the effect of the dead load in stiffening the bridge is neglected, while in the deflection theory it is taken into consideration and exerts an important influence on the results.

The main considerations of the deflection theory can be explained without recourse to mathematical symbols. The suspended trusses together with the cables, towers, and anchorages form an elastic structure acting as one system, that is, an external cause affects the members of the entire system elastically and on its passing the system is fully restored to its original condition. Such causes in bridges are usually live loads, wind pressures, and temperature changes.

* "The Towers, Cables, and Stiffening Trusses of the Bridge over the Delaware River, between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

In the analysis of stresses and deflections of common truss systems the assumption is made that the elastic system when subjected to the specified causes undergoes deformations which are so small that the resulting distorted system may with sufficient accuracy be treated as undistorted and the original dimensions may be retained in the stress computations. This holds true for most stiff frames and the usual bridge trusses, but it is not correct for long-span suspension bridges. The deformations of the original geometric figure of suspension bridges are of such magnitude that serious differences in stresses result if the original dimensions are adhered to in the computations.

The reason for this phenomenon is that a suspension bridge or an arch, when erected, forms an elastic system in equilibrium under its dead load; that any live load or its equivalent causes a distortion of the original figure and establishes a new system in equilibrium. The main factors in the distortion are a change in length of the cable and a distortion of the stiffening trusses, likewise of the cable curve.

Prior to the connection of the individual truss members to form a continuous truss and prior to connecting the truss ends to their reaction points the cables form an equilibrium polygon subject to dead load concentrations as transmitted by the suspenders and the weight of the cable itself. After the connection is made an additional or live load on the system disturbs the old equilibrium and a new one tends to establish itself. This is accomplished by a deformation of the original polygon and truss. The truss deflects and this action is transmitted through the reactions of the suspenders to the cable. The latter thus forms a new polygon. The deformation is resisted by the elastic stiffness of the truss and by the work required to distort the original equilibrium polygon into the new one.

The stiffness of the truss is determined by its moment of inertia and the modulus of elasticity of its material. The stiffness of an equilibrium polygon on the other hand is a function of the forces acting upon it. The greater they are the more work will be required to distort the polygon. Therefore the greater the dead load of the bridge acting on the cable the stiffer the cable will be. The influence of the dead load on the stiffness of suspension bridges thus becomes apparent.

Let the following serve as an illustration by a rough example. Assume a span of 2 000 ft. with a versine of 250 ft. Such a span may be assumed to weigh 20 000 lb. per lin. ft. The horizontal pull in the cable will thus be $H = 40\,000\,000$ lb. Let a live load be placed on the bridge, which will cause a downward deflection at mid-span of, say, 5 ft. The newly distorted cable curve will thus be 5 ft. lower at its crown; in other words, its versine will be 255 ft. The new horizontal pull due to the dead load will become $H = 39\,216\,000$ lb., or about 784 000 lb. less. As is well known, the general expression for the bending moment at any point of the stiffening truss is given by the moment of the live load for a simple beam less the product of H times the ordinate of the cable curve at that point. Thus, multiplying the reduction in H (784 000 lb.) by 255 ft. gives a relieving moment of 200 000 000 ft-lb. which will reduce the bending in the truss. The effect of the live load

also adds to the reduction. The magnitude of the reduction in the bending moment is a sufficient illustration of its importance both as to the stresses involved and as to the economy resulting.

From this example it becomes apparent that the heavier the bridge, the greater the reduction of the stresses in the truss will be. This emphasizes the common experience that the heavier a suspended chain, the more difficult it is to distort. The dead load effect was known, of course, to the early builders of unstiffened suspension bridges, but was forgotten afterward by the introduction of the rigid and elastic theories.

The mathematical treatment consists in establishing the general equation of the curve of flexure of the truss and in deriving an expression of condition for the horizontal component of the cable pull, H, by equating the work of the external forces acting on the cable to its internal work of deformation. The expression for H is solved by successive numerical approximations. This is the time-consuming part of the computations. On the other hand, the determination of the moments and shears takes but little time. Moreover, the deflection of the truss at any point can be computed very readily. As explained, the entire theory is based on the deflection of the system. It has been thought appropriate, therefore, to name it the deflection theory.

The Manhattan Bridge, in New York, and the Delaware River Bridge, in Philadelphia, Pa., have been computed and designed in accordance with the deflection theory. In the case of the Delaware River Bridge, the speaker has made a comparison of the two theories by determining the bending moments and shears in the stiffening trusses for the identical structure. The results are given in Figs. 23, 24, 25, and 26, which show the moment and shear curves for the center and side spans plotted for both theories.

Quoting directly from the speaker's paper previously mentioned:

"The areas enclosed within the two curves represent the measure of the theoretical saving effected in the stiffening trusses. For the center span the application of the elastic theory would require theoretically 55 per cent. more material for the chords and 43 per cent. for the diagonal web members; for the side spans it would require 40 per cent. for the chords and 26 per cent. for the web members.

"In the actual design of the truss members certain minima sections had to be used for practical considerations. These reduce the theoretical saving to the actual saving of 51 per cent. for the chords of the central spans and 43 per cent. for its web members. The actual saving for the chords of the side spans is 38 per cent., and for the diagonal web members 24 per cent.

"The total saving in steel is 3 211 tons, or 42 per cent., of the total amount used in the trusses proper.

"Based on the contract prices obtained for the stiffening trusses, the cost of the 3 211 tons is:

Special steel	5 024 000 lb.	@	8.75 c.	$440 000
Silicon steel	1 244 000 lb.	@	7.85 c.	98 000
Carbon steel	154 000 lb.	@	6.90 c.	10 000
Total				$548 000

The above sum of $548 000 represents the direct saving effected on the trusses.

"The increased weight of trusses would, however, require additional cable wire and suspender ropes to sustain it, which amounts to $206 000. Another

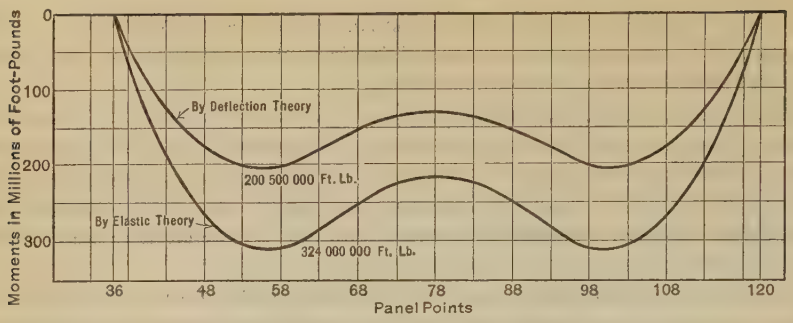

FIG. 23.—COMPARISON OF MOMENTS IN CENTER SPAN.

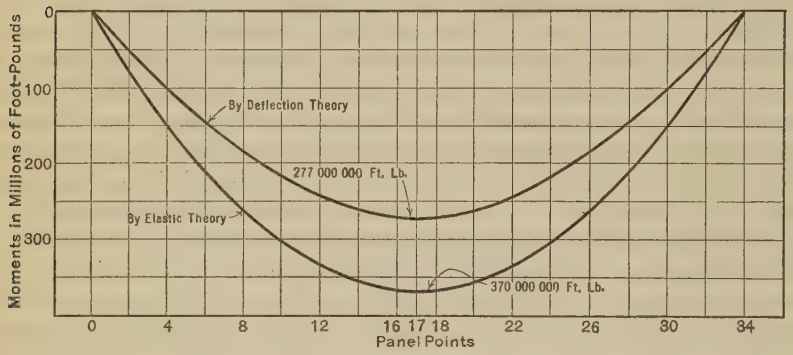

FIG. 24.—COMPARISON OF MOMENTS IN SIDE SPANS.

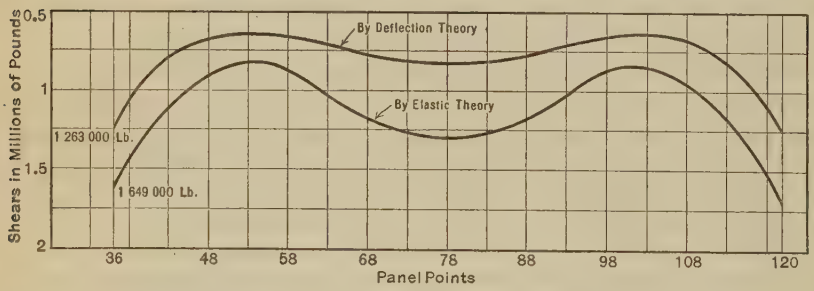

FIG. 25.—COMPARISON OF SHEARS IN CENTER SPAN.

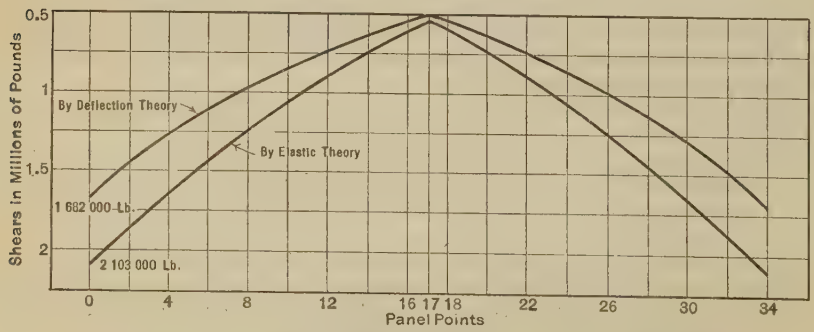

FIG. 26.—COMPARISON OF SHEARS IN SIDE SPANS.

$96 000 would be required by the towers and anchorage bents. A total of $302 000 would then be added to the saving on the trusses. The total saving effected by the use of the deflection theory would thus amount to $850 000."

In the Manhattan Bridge the reduction from the requirements of the elastic theory was to about 74% of the theoretical chord areas.

The speaker did not make any deductions as to the law of variation of one theory from the other or as to the percentage of reduction for different spans under various dead and live loads. He intended to show the error involved in the use of the elastic theory and to prove that it is obsolete and should be abandoned. Mr. Waddell had evidently made his computations based on the elastic theory and afterward, realizing that it is in error, reduced the results by the single comparison available to him.

Altogether the assumptions made by the author and the results presented by him should be treated broadly. It is a question whether he ever intended them to be taken as rules for design. In other words, whether the author allows for the cables 70 000 lb. per sq. in., and others may go as far as 80 000 and even 100 000 lb., whether the same unit stresses will be used for nickel steel, or whether the assumed live load will persist, the general curves and figures will be found to be broadly approximate. Certainly they will be close enough for what any engineer of experience may expect to get in two or three hours' work.

Francis P. Witmer,* M. Am. Soc. C. E. (by letter).—The writer is particularly impressed with the author's conclusions as influenced by the very startling claim of saving in weight of stiffening trusses, through the use of the "deflection theory" instead of the "elastic theory" in their design. Such a saving as 36% in weight of trusses in the case of a large bridge seems hardly credible, and the writer is led to question the correctness of this percentage.

The explanation of the deflection theory given by Mr. Moisseiff is a necessary supplement to the paper itself. Referring to his example, it was asserted that "multiplying the reduction in H (784 000 lb.) by 255 ft. gives a relieving moment of 200 000 000 ft-lb. which will reduce the bending in the truss". Here the writer fails to follow the argument; he cannot see the justification of thus multiplying the difference between the values of the horizontal dead load pull in the cable for two different versines by either of these versines in order to obtain the value of such a relieving moment. The total moment at the center of the stiffening truss from the dead load cable stress before the live load was applied would be,

$$40\,000\,000 \times 250 = 10\,000\,000\,000 \text{ ft-lb.}$$

and the moment from the dead load cable stress after the live load was applied would be,

$$39\,216\,000 \times 255 = 10\,000\,000\,000 \text{ ft-lb.}$$

As these moments are equal, the relieving moment from dead load really appears to vanish entirely.

* Director, Civ. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

The writer has made no quantitative comparison between the actual truss stresses in any given case under the two methods of calculation, and is in no position to question results which others may have obtained by a complete application of each method throughout a specific design. If, however, the percentage of saving alleged in the paper has resulted from a consideration of a dead load relieving moment obtained as described, he seriously doubts whether any such percentage of saving or, in fact, any saving of appreciable proportions will result by the use of the "deflection theory" instead of the "elastic theory."

V. A. Eberly,* Assoc. M. Am. Soc. C. E. (by letter).—Designs for highway suspension bridges for spans of less than 500 ft. and proportioned for modern concrete floor-slab construction are probably not placed in bidding competition with other types frequently enough by bridge engineers. The extension of Mr. Waddell's curves down to a 250-ft. main span would prove of considerable interest.

It has been shown† by O. H. Ammann, M. Am. Soc. C. E., that suspension spans of 300 ft., or less, compete favorably in cost with truss spans. The writer has detailed a two-hinged, 250-ft. suspension span (the cables carrying no side spans) with a concrete floor-slab. The roadway slab measured 251.1 ft. long by 19.83 ft. wide, which is an area of 4 980 sq. ft. This bridge was estimated to cost $40 800, which is an equivalent cost of $8.20 per sq. ft. The design called for adjustable hangers, and two shop-assembled wire cables on each side of the roadway. There were no sidewalks.

In designing these shorter suspension bridges the writer believes (*a*) that non-adjustable hangers should be used; (*b*) that if wire cables are used, they should be carried back into the anchorages, eliminating the use of bridge sockets; (*c*) that eye-bars will prove better for long-period maintenance; (*d*) that the towers should be transversely battered inward at the top, to give more roadway clearance at the elevation of traffic movement; and (*e*) that the cables or eye-bar tension members should be cradled in plan and also to afford more satisfactory roadway clearances.

The embedded anchorages designed for the 250-ft. span suspension bridge differed from the usual anchorages in which the tension member passes in a sweeping curve over the upper part and down through the rear to the base. In the ordinary layout overturning resistance is depended on to hold the cables unyielding, and high forward toe pressures and high front face pressures in the upper region of the anchorage are induced—the latter particularly, if it is attempted to support the upper sweep of the cable arc on a narrow wall, such as a parapet wall.

Under the method adopted by the writer (shown diagrammatically in Fig. 27 (*b*)) the entire anchorage, in order to yield, would move forward without rotating. Its resistance is derived from (1) bottom friction on its base; (2) friction on the sides; and (3) the resistance of a wedge-shaped section of earth as determined by the following consideration.

* Bridge Designer, Ohio State Highway and Public Works Dept., Columbus, Ohio.
† *Engineering News-Record*, June 21, 1923.

*Resistance of Earth in Front of a Retaining Wall to Sliding.**—In this deduction the cohesion of the earth is neglected, friction only being considered.

Let H = horizontal resistance to sliding of earth in front of wall of unit length.

w = unit weight of earth.

ϕ = friction angle of earth.

i = angle of slope of earth in front of wall.

d = depth of wall below surface of earth in front.

α = angle with horizontal of plane of least resistance to sliding.

W = total weight of earth in sliding wedge of unit length of wall.

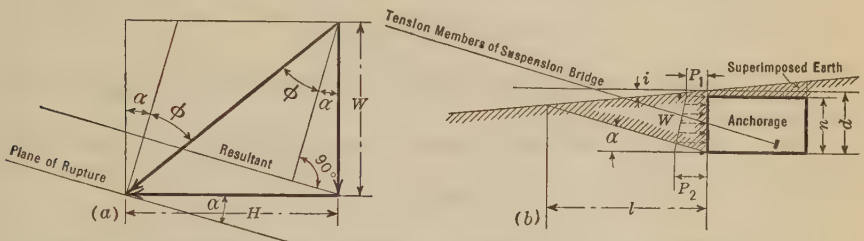

FIG. 27.

Then, from Fig. 27 (*b*):

$$W = \frac{1}{2} w d l = \frac{\frac{1}{2} w d^2}{\tan \alpha + \tan i}$$

From Fig. 27 (*a*):

$$H (\cos \alpha - \sin \alpha \tan \phi) = W (\sin \alpha + \cos \alpha \tan \phi)$$

$$H = \frac{W (\tan \alpha + \tan \phi)}{1 - \tan \alpha \cdot \tan \phi} = W \tan (\alpha + \phi)$$

$$= \frac{1}{2} w d^2 \frac{\tan (\alpha + \phi)}{\tan \alpha + \tan i} \quad \ldots\ldots (2)$$

In order to find the plane of least resistance to sliding, equate $\frac{\delta H}{\delta \alpha}$ (obtained by differentiating Equation (2) with H and α as variables), to zero; then,

$$\cos \alpha \sqrt{1 - \cos^2 \alpha} = \left(\cot \phi - \frac{\tan i}{2 \sin^2 \phi}\right) \cos^2 \alpha - \frac{1}{2} \cot \phi \quad \ldots\ldots (3)$$

Letting

$$A = \cot \phi - \frac{\tan i}{2 \sin^2 \phi}$$

Equation (3) reduces to:

$$\cos \alpha = \sqrt{\frac{1 + A \cot \phi}{2 (1 + A^2)} \left[1 + \sqrt{1 - \frac{(1 + A^2) \cot^2 \phi}{(1 + A \cot \phi)^2}}\right]} \quad \ldots\ldots (4)$$

* Deduced by A. W. Zesiger, M. Am. Soc. C. E., who has consented to this presentation.

In order to find for what value of i, $\alpha = 0$, let $\alpha = 0$ in Equation (3) and solve for tan i which reduces to,

$$\tan i = \frac{1}{2} \sin 2\phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

α is positive for values of i greater, and negative for values less, than that obtained from Equation (5).

When $i = 0$, Equation (4), reduces to,

$$\cos \alpha = \sqrt{\frac{1}{2}(1 + \sin \phi)} = \cos\left(45° - \frac{1}{2}\phi\right) \ldots\ldots\ldots\ldots (6)$$

or,

$$\alpha = 45 - \frac{1}{2}\phi$$

It also should be shown that H must not exceed the horizontal heaving power of the earth (Fig. 27(b)).

Let p_1 and p_2 represent the horizontal heaving power of the earth. Then the allowed H must not exceed $\frac{1}{2} n (p_1 + p_2)$.

The appearance of the suspension bridge stands foremost among steel bridges. The complete simplicity of the new self-suspending bridges at Pittsburgh, Pa., presents a bridge architecture of unequalled beauty. With a concrete-slab floor system the problem of the rigidity of the highway suspension bridge is solved to a great extent.

Horatio P. Van Cleve,* M. Am. Soc. C. E. (by letter).—Few members of the Society have ever designed or estimated costs on a suspension bridge that has been actually constructed. Most engineers, however, are interested in engineering costs as such, and in simple methods of arriving at the quantities from which those costs can be derived.

According to the Synopsis of the paper: "Its object is to enable any engineer of good, general experience to compute, in two or three hours, * * * the total cost of any * * * suspension bridge * * *." For the purpose of preliminary estimate, as a means of comparison with other types of design, the desired object seems to have been attained, and the curves given should prove very convenient; but probably in many cases substructure and approach quantities would be subject to considerable modification if a closely approximate estimate of cost were desired. The same could probably be said of superstructure quantities, although to a less degree.

Although engineering cost analyses are usually quite ephemeral and out of date soon after they are printed, the quantitative curves available in this paper should make it a fairly simple matter to revise the cost curves as conditions may dictate.

Rising costs of engineering erection labor have accompanied fairly uniform or falling prices of the chief construction materials (Fig. 28). Twenty years ago the average cost of structural steel shapes at Pittsburgh was $1.70. Dur-

* Chf. Engr., J. Edward Ogden Co., New York, N. Y.

ing the years of heavy war demand this cost rose rather sharply but now it is $1.95. This speaks volumes for the increased efficiency of manufacturing methods that have made such a condition possible in spite of the rising cost of labor. Since 1913, Portland cement has a little less than doubled in cost and is now falling somewhat, while Southern pine is double the price obtaining in 1913.

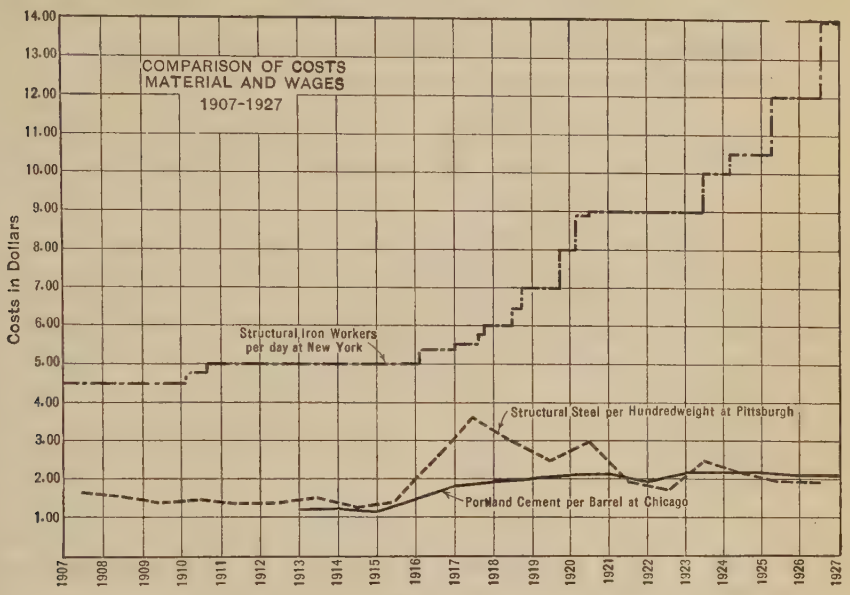

FIG. 28.

As to erection labor, however, it is a different story. In 1907 structural iron workers in New York City were receiving $4.50 per day. In 1927 the rate is $14.00. There is little or no increase in efficiency, however, and the day of decreased cost in this line does not seem to be in sight. Agitation for a 5-day week seems to be the vogue, and if it comes it will certainly mean a general increase in rates to offset the decrease in working time.

The increase in the wages of iron workers during the last 10 years has been 16.4% per year (Fig. 28). This is exceeded for the concrete workers who have had an average increase of 18.3% per year over the 10-year period. It is probably fair to assume that the present cost of erection labor for average construction is not much less than 40% of the total cost. Assuming that erection wages increase for the next five years at the rate applying to the iron workers of New York City over the 10-year period just ended, and that material prices remain as they are at present, each construction dollar, instead of being represented by $0.40 for erection and $0.60 for all other costs, will be composed of $0.55 for erection and $0.45 for all other costs.

Of course, in some items of bridge construction the erection labor cost is far more than 40% of the total. This is particularly true of concrete piers. It hardly seems possible that erection labor can continue to advance at the

average rate of the last ten years, for a general slowing down in building operations would result.

Assume that all erection labor were to increase at the rate of 16.4% per year, based on the present rates, and continued for five years. How would it affect the unit prices given in the paper? Table 4 is an attempt to separate erection costs from the totals given by the author, which are assumed to be correct, and to compare them with new units based on the assumed increase.

TABLE 4.—Revised Unit Prices for Advancing Labor Costs.

Materials. (1)	Material delivered. (2)	Erection labor. (3)	Unit price. (4)	New erection labor Column (3) × Column (5). (5)	New unit price, Column (2) + Column (5). (6)	Percentage of increase, Column (6) over Column (4). (7)
Carbon steel	$0.045	$0.025	$0.07	$0.045	$0.09	28.6
Silicon steel	0.065	0.025	0.09	0.045	0.11	22.3
Nickel steel	0.075	0.025	0.10	0.045	0.12	20.0
Wire cables	0.15	0.03	0.18	0.055	0.205	13.9
Floor-slab, including reinforcement	8.00	27.00	35.00	49.20	57.20	63.4
Pavement	1.60	1.20	3.00	2.18	3.78	26.0
Electric-railway track	3.60	2.40	6.00	4.37	7.97	32.8
Shafts of piers	7.00	13.00	20.00	23.70	30.70	53.5
Shafts of anchorages	7.00	8.00	15.00	14.60	21.60	44.0
Bases of pneumatic piers	7.00	28.00	35.00	51.00	58.00	65.6
Bases of pile anchorages	7.00	23.00	30.00	41.90	48.90	63.0
Piles below anchorage bases	.25	1.00	1.25	1.82	2.07	65.5

From the percentages of increase (Column (7) of Table 4), it is evident that advancing costs of erection labor are a matter of considerable moment to the engineer and contractor as well as to the general public.

Wendell P. Brown,* M. Am. Soc. C. E. (by letter).—Few bridge engineers have the opportunity in the regular course of their professional activities to determine the cost of such long-span structures. Therefore, these data will be valuable as a reference and check on similar estimates for those who are required to make them.

The use of unit costs per square foot of floor of structure for preliminary estimates is a satisfactory and fairly accurate method, but it should be done by those who are familiar with the use of such data, and experienced in their application to the various conditions affecting the particular estimate under consideration. Although not in a position to confirm the estimates for structures of such unusual length of span, the writer judges from the unit prices used for the different classes of materials that the costs per square foot are conservative and sufficient.

The cost of lighting, however, seems rather low, except for a very simple type of installation, but this item is a small proportion of the entire cost and the total would be but little affected. The use of tower encasement, while permissible, and perhaps desirable under certain conditions, is of somewhat questionable value, either from the standpoint of appearance or of practicability.

* Pres. and Chf. Engr., The Wendell P. Brown Co., Cleveland, Ohio.

WILLIAM G. GROVE,* M. AM. SOC. C. E.—As the author requests full discussion of the paper from all angles and as the eye-bar type of cable was eliminated from consideration after a brief reference to cables, this discussion is presented for the purpose of comparing the eye-bar and wire types of cable and of attempting to show some of the possibilities of eye-bar cable construction.

In Table 1, five different span lengths, ranging from 1 000 to 5 000 ft., are considered, and the approximate sag ratios, that is, the ratio of the cable sag to the span length, from the assumed dimensions are, as follows:

1	to	10	for	the	1 000-ft. span
1	"	11	"	"	2 000-" "
1	"	11½	"	"	3 000-" "
1	"	12	"	"	4 000-" "
1	"	12	"	"	5 000-" "

For the eye-bar type of cable these ratios seem somewhat small and a more economical design would result by increasing the actual amount of cable sag for each of these five examples. However, for the sake of uniformity the discussion will be based on these ratios, for which the length of the cable is about 3% greater than the span length for the 1 000-ft. span and about 2% greater for the other spans. Also, for these ratios, the stress at the tower is about 8% greater than that at the center of the span for the 1 000-ft. span; about 7% greater for the 2 000-ft. span; and about 6% greater for the 3 000, 4 000, and 5 000-ft. spans. With the eye-bar type of cable it is therefore possible and economical to adjust the sectional area of the cable along the arc so as to provide for this variation of stress. The average area will be about 2% greater than that at the center of the span for all the span lengths given.

Assuming an eye-bar cable of any cross-section, say, 100 sq. in., at the center of the span; add 2% to allow for the average area of the cable; add 3% to allow for the increase of the arc of the cable over the span length; and, finally, add 15% for details such as eye-bar heads and pins. Then, for a 1 000-ft. span, the cable under its own weight only, will have a unit stress of approximately 5 000 lb. per sq. in.

Allowing 2% increase for average cable area, 2% for increase of arc over span length, and 15% for detail, the unit stress, in a cable under its own weight only, will be approximately,

11 000	lb.	per	sq.	in.	for	a	2 000-ft.	span	
18 000	"	"	"	"	"	"	3 000-"	"	
24 000	"	"	"	"	"	"	4 000-"	"	
30 000	"	"	"	"	"	"	5 000-"	"	

Therefore, it is easily seen that ordinary steel (16 000 lb. per sq. in.) cannot be used for a 3 000-ft. span, because the stress in an eye-bar cable under its own weight without any truss, floor, or live load, is 18 000 lb. per sq. in., or more than is used for that grade of steel. Furthermore, heat-treated steel having an elastic limit of 50 000 lb. per sq. in., cannot be used in a span of 5 000 ft., because the stress in an eye-bar cable under its

* Asst. Engr., Am. Bridge Co., New York, N. Y.

own weight, without any truss, floor, or live load, is 30 000 lb per sq. in. and only 32 000 lb. is permissible in this grade of steel.

In order to compete with steel wire for long-span suspension bridges, there has been developed the high tensile strength heat-treated eye-bar steel, having a guaranteed minimum elastic limit of 75 000 lb. per sq. in. and a guaranteed minimum ultimate strength of 105 000 lb. per sq. in. The first application of this high tensile strength heat-treated steel was in the Florianopolis Bridge in Brazil, designed by H. D. Robinson and D. B. Steinman, Members Am. Soc. C. E., and the results of the thirteen full-sized eye-bar tests on that bridge, showed an average elastic limit of approximately 85 000 lb. and an average ultimate strength of 123 000 lb. per sq. in. On that basis it was considered satisfactory to use a working stress of 50 000 lb. per sq. in.

On all wire cable suspension bridges, to the speaker's knowledge, the area of the cable is constant from tower to tower. Therefore, the stress at the tower determines the entire section so that the area of a wire cable will be 8% greater than required by the stress at the center for a 1 000-ft. span, 7% greater for a 2 000-ft. span, and 6% greater for the other three spans. On wire cables, 8% should be added for details to cover clamps and cable wrapping.

If the cost per pound of the wire cable, complete in place, including cable clamps and the small wire protective wrapping, for a 1 000-ft. span is multiplied (1) by $\frac{1.08}{1.15}$ to allow for details; (2) by $\frac{1.08}{1.02}$ to allow for the fact that the section of the wire cables is constant between towers while the eye-bar cables have varying thicknesses; and (3) by the ratio of $\frac{50\,000}{70\,000}$, the denominator being the unit stress allowed by Mr. Waddell for wire cables, there results a cost per pound which the eye-bar cable must approximate in order to compare favorably with a wire cable. The product of these factors is about seven-tenths. Following the same procedure for the other spans and using the proper factors in each case, it is found that the products of the three factors vary from two-thirds to seven-tenths.

While this method is satisfactory for the shorter spans it is not sufficiently accurate for the longer spans because it does not take into consideration the the fact that part of the unit stress in the cable is used to support the cable itself. This part is only 5 000 lb. per sq. in. for the 1 000-ft. span, while it becomes 30 000 lb. for the 5 000-ft. span. The former figure is only a small percentage of the unit working stresses, while the latter figure is a large percentage, and by increasing the span a length will be reached for which the entire working stress of 50 000 lb. will be used to support the cable itself, this length being approximately 8 200 ft.

Using the unit stress ratio such as to allow for the fact that the cable must support itself, the revised products of the three factors are:

0.69 for the 1 000-ft. span
0.65 " " 2 000-" "
0.60 " " 3 000-" "
0.54 " " 4 000-" "
0.47 " " 5 000-" "

Assuming $0.18 per lb. (given by Mr. Waddell) as the cost of the wire cable in place, including the clamps and wire wrapping, it is found that the cost of an eye-bar cable in place should approximate:

$0.124	per	lb.	for	the	1 000-ft.	span
0.117	"	"	"	"	2 000- "	"
0.108	"	"	"	"	3 000- "	"
0.097	"	"	"	"	4 000- "	"
0.085	"	"	"	"	5 000- "	"

in order for the eye-bars to compare favorably with wire; but other factors affect this comparison.

While it is true that the eye-bar cable will require somewhat heavier towers and anchorages than the wire cable, due to the fact that it contains more steel, there is another factor that offsets this increase. In the modern wire cable suspension bridges, the type of erection equipment used for the construction of the cables has been materially different from that used for the erection of the remainder of the main and side spans. In the case of an eye-bar cable suspension bridge it is possible to use the same erection equipment for the main span as that used for the erection of the eye-bars.

For the Florianopolis Bridge the entire main span and eye-bar cables were erected by the overhead method, that is, by small trolleys running on erection ropes. The trolley used for the eye-bar erection weighed 20 000 lb. and the two small trolleys used for the erection of 1 000 members of the main span, trusses, floor-beams, stringers, bottom laterals, top laterals, top struts, and sidewalks brackets, weighed only 5 000 lb. each.*

This brief discussion is presented merely for the purpose of showing that eye-bar cables using heat-treated steel having an elastic limit of 75 000 lb. per sq. in., compare favorably with wire cables for all except extremely long spans. It is not a fair comparison, however, to think of the cable alone; the entire structure in place must be considered. Furthermore, each individual bridge must be considered on its own merits. It is impossible to determine by estimate alone which type is the cheaper. There is only one sure way to find out, namely, to ask for comparative bids, as was done for the Florianopolis Bridge, where the contract price of the eye-bar cable in place was lower than the lowest bid or estimate received on either wire cable or wire rope cable in place, and the cost of the bridge was inside the estimate. Finally, the overhead trolley method of erection is safe because the Florianopolis Bridge with all its novel features was successfully erected 6 000 miles from the United States without the loss of a single human life.

M. Godard† (by letter).—Europe, and particularly France, has no equivalents of the American long suspension bridges. The lower limit of 500 ft., as shown in the tables and on the diagrams of Mr. Waddell's paper, is almost the upper limit for the suspension bridges commonly used in Europe.

This paper, therefore, forms for French engineers a valuable document to which they can add little drawn from their personal experience. For his

* For a complete description of the design and erection of the Florianopolis Bridge, see *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 707.

† Prof. of Bridge Eng., l'Ecole Nationale des Ponts et Chaussées, Paris, France.

part, the writer has gathered from it interesting information which will be of use in his lectures at the L'Ecole Nationale des Ponts et Chaussées.

The unit stress adopted for the wire, which corresponds to 108 kg. per sq. mm. of cross-section, appears to be very high. One wonders what tolerance has been fixed to take account of the fact that the various parallel wires assembled one with another are not strained in an absolutely equal fashion. In France, in suspension bridges with parallel wires, only 80% of the total cross-section of the wires composing the cable is taken into account for the calculation of the resistance.

In studying the effect of wind, Mr. Waddell indicates that according to certain authorities, in long spans, the effect of wind load on the trusses and their laterals is scarcely worth considering, owing primarily to the very mass of the cables and to the rigidity of the large concrete platform forming the floors of highway bridges.

Again, the diagrams of wind loads per foot of span show that the load to be included in the calculations is supposedly much less for large structures than for structures having medium or short spans. The writer confesses that he does not understand this very well. The action of the wind on suspension bridges can be calculated, theoretically, exactly as for bridges of other systems, for example, tubular bridges. The type of floor considered by Mr. Waddell is one with two horizontal stiffening trusses, that is, a true suspended tubular bridge. In a structure of this kind, subjected to the action of the wind the cables have a tendency to return horizontally to their original position, this tendency increasing with the span. From this viewpoint it is conceived that the horizontal action to be taken into account in calculating the horizontal stiffening trusses (such action being the difference between the effect of the wind and the tendency of the cable to return horizontally) is the feebler the greater the span and actually may be considered as almost constant, starting from a certain span. This, however, is not true of the ribs; in the calculation of these the returning action of the cables plays no part and the effect of the wind needs to be considered by itself and in its full intensity. This returning action of suspension bridge cables is quite difficult to calculate exactly. However, Inspector-General Pigeaud has described* a method that permits the calculation to be made with all the necessary accuracy.

O. H. Ammann,† M. Am. Soc. C. E.—It has been stated that engineers design superstructures to a mathematical nicety and then trust to good luck that the foundations will carry them safely.

In addition to the large and varied margin of safety in designing foundations, this attitude applies more or less also to superstructures, in that the mathematical nicety with which stresses are determined is a mere cloak to hide the somewhat crude methods used in designing these structures.

To-day, the design of superstructures is as much a matter of sound judgment as the design of sub-surface structures. Of course, this refers to the broader aspect of design, freed from the trammels of so-called standard

* *Le Génie Civil*, July 25, 1925.

† Bridge Engr., The Port of New York Authority, New York, N. Y.

specifications and conventional practice. Standard practice fixes permissible stresses which in certain members could be exceeded by 100% with perfect safety, while in others no one would dare go beyond an excess of 30 or 40 per cent.

Even in so-called statically determinate structures stresses are determined to a varying degree of accuracy sometimes not within 100% of the true amounts. The difference between the true and calculated stresses is called secondary stress and provided for in a widely varying manner.

Considering, further, the wide field open to the engineer in choosing the type and proportions of a structure, and the variations so largely influenced by local conditions; making due allowance, also, for the freedom which the designer should enjoy in giving expression to his esthetic tastes; and considering that all these influences are accentuated in extraordinary structures, it must be admitted that there is ample room for the exercise of individual judgment and of broad-minded tolerance in solving a problem of such complexity as a large suspension bridge.

Thus, although at variance with some of the author's conclusions, the speaker has no intention to question the value of the paper, for it is through such investigations and discussions that the profession will benefit. No one who has not, for himself, delved into such economic studies, can acquire the freedom of judgment essential to all broad planning.

It would lead too far afield to enter into a discussion of all features of long-span suspension-bridge design that influence the cost of such structures. The speaker desires to single out the one feature of preponderant economic influence in long suspension spans in respect to which he differs most widely with the author, namely, the design of the vertical and lateral stiffening systems.

Practically all the rationally built suspension bridges dating back to the end of the Eighteenth and the first half of the Nineteenth Centuries were of the flexible type. Many of them are still in useful service. Some had no special stiffening system, sometimes not even lateral bracing. The majority belonged to what would be classified to-day as short spans; in comparison with present-day structures, they were of very light construction. For the loads which they were designed to carry many of them have given excellent service and it is to their credit that they withstood the later heavier loads and severer traffic conditions longer and better than other, more rigid types.

Several excellent illustrations of the character and behavior of these early bridges have been given by the late William Hildenbrand, M. Am. Soc. C. E.* Such extreme deflections as he records are of course objectionable from the point of view of modern traffic, and it became apparent that the unstiffened cable or chain of light construction and short span did not possess sufficient rigidity for the heavier loads and more exacting traffic requirements. This led to the introduction, in the second half of the Nineteenth Century, of the so-called stiffened suspension systems and to widespread elimination of the suspension type for short spans.

* *Transactions*, Am. Soc. C. E., Vol. XLVIII (1902), p. 436 *et seq.*

French engineers, however, have continued to recognize the advantage of partial flexibility, with the result that, in France and in the French Colonies, are found many excellent and economically built modern suspension bridges of moderate span. In Central Europe, however, and of late years in America, the rigid system, and its theory, have become so firmly implanted in the minds of some engineers and theorists that they appear to ignore its limitations and the possibilities and virtues of the flexible system.

In some modern textbooks the flexible system and the theory adapted to it seem to have been relegated to the past, and certain economies of the suspension bridge have been developed entirely on the theory of the rigid truss, without regard to span length, weight, and character of load. A peculiar outcome of this practice is the repeatedly expressed opinion that the proportioning of the stiffening system is dependent solely on the live load, irrespective of the dead weight. If this view were correct, it would involve progressive increase in weight of these trusses, with increasing span and load.

The speaker has called attention* to this erroneous view and has given the increase in dead weight of modern bridges, due mainly to heavier floor construction, as a factor favoring the suspension bridge in permitting more economical design of the stiffening system; but it is particularly in the field of modern long-span suspension bridges, with their enormous dead weights, that great savings can be effected by proper design of that system.

In recognition of the inadequacy of the theory of the rigid truss, when applied to a more or less flexible system, a number of so-called "exact" methods have been developed. However, even the results of these exact theories become increasingly discrepant and unreliable with increasing flexibility of the system. In order to form a clear conception of the source of these discrepant views and theories, without entering into the intricacies of the theories of the suspension system, it is necessary only to consider the incongruous behavior of the two elements composing the system, namely, the flexible cable and the elastic truss. In action these two carrying elements are radically different in several respects:

First.—The elastic line of the independent truss due to any given load does not coincide with the distorted catenary of the flexible cable, either in form of curve or in amount of deflection. For a comparatively rigid truss and large ratio of live load to dead weight the distortions of the cable are much greater than the deflections of the truss, while the reverse is true for a flexible truss and small ratio of live to dead load.

Second.—The elastic deflections of the truss increase proportionately with the load, while the vertical distortions of the cable increase at a gradually decreasing ratio to the load. Herein lies a source of added safety of the flexible truss against failure due to increase in loads, such as has occurred in a number of suspension bridges with rigid trusses.

Third.—The elastic line of the truss under live load is independent of its own weight, or the dead weight imposed on it, whereas the distortions of the flexible cable are the smaller the greater its own weight and that suspended

* *Engineering News-Record*, June 21, 1925.

from it. For small ratios of live to dead load, the distortions of the cable from the same live load are almost inversely proportional to the dead weight.

The stiffened suspension bridge is a combination of these two incongruously behaving elements. It is evident that they must influence each other to a varying degree, and that their joint action must be a complex, mathematically inexpressible, compromise. The more rigid the truss and the lighter the cable, as is the case in short spans, the more will the truss impress on the combined system its own characteristics. The more flexible the truss and the heavier the cable, on the other hand, the more will the characteristics of the unstiffened cable prevail, and the more, therefore, can the dead weight of the structure be utilized in securing a desired degree of rigidity.

When the dead weight becomes so great that the deformations of the unstiffened cable under actual load conditions remain well within permissible limits, then it becomes possible to dispense with a stiffening system altogether. There are other factors, however, such as flatness of the catenary, shortness of the side spans, and relatively large sectional area of cable or chain, which may combine with the dead weight in producing an ample degree of rigidity without the aid of a stiffening system. The length of center span at which it becomes feasible to dispense with such a system, therefore, depends on all these factors.

There is another important factor. It is recognized that short spans require a rigid system mainly to prevent excessive oscillations under impact from modern high-speed vehicles, while in large, heavy bridges the permissible deflections are governed solely by the maximum permissible gradient of the floor. It may be demonstrated easily that in long-span suspension bridges of the flexible type, the most unfavorable and improbable load and temperature conditions will not produce gradients that are excessive for modern electric traction and vehicular traffic.

Does it seem consistent, therefore, to apply the same methods and theories to the proportioning of stiffening systems for all spans? And is it not evident that the method properly applicable to short spans must become increasingly wasteful with added weight and length of span?

The author refers to the Delaware River Bridge, in Philadelphia, with a span of 1 750 ft., noting that by the application of an exact or "deflection theory" as much as 40% of the material in the stiffening trusses was saved. It might perhaps be more appropriately stated that this saving was due to the comparative flexibility of these trusses, together with the application of an exact theory.

In designing the Hudson River Bridge between Fort Washington, New York City, and Fort Lee, on the New Jersey side, which is to have a central span of 3 500 ft., and side spans of only 650 ft., and is to carry highway and electric railway traffic, the speaker found that these favorable span conditions permitted the use of flexible trusses of a height-to-span ratio of only 1:120, so that fully 80% of the material required for a rigid truss, proportioned in accordance with the common theory, could be saved.

Considering that in such a large span every pound of steel unnecessarily used in the stiffening trusses represents merely ballast, adding nothing what-

ever to strength or durability, and that this excess steel means an equivalent additional waste in the cables, towers, and anchorages required to carry it, it is evident that such wasteful proportioning may run into many millions of dollars.

The Hudson River Bridge is designed for a traffic capacity approximately equivalent to that of the Manhattan Bridge, New York City, or for a working live load of 8 000 lb. per lin. ft. It has a span length 2.38 times that of the Manhattan Bridge. The stiffening system of the older bridge, which is comparatively flexible and has been properly proportioned in accordance with the deflection theory, weighs 7 200 lb. per ft. of bridge. A stiffening system of similar proportions in the Hudson River Bridge should, in accordance with the common theory of the rigid system, weigh roughly $7\,200 \times 2.38 = 17\,200$ lb. per lin. ft. In the design as developed, with shallow flexible trusses, giving at least the same relative rigidity as the more rigid system in the Manhattan Bridge (due to the relatively much greater dead load, the longer center span, and the shorter side spans of the Hudson River Bridge) the weight of the stiffening system is only 2 500 lb. per lin. ft. This represents, therefore, a saving of about 14 700 lb. per lin. ft. of bridge, or approximately 35 000 tons in all. Being based on the use of high alloy steel in the chords, this saving represents at least $6 000 000. To this must be added an equivalent amount due to saving of material in the suspenders, cables, anchorages, and towers required to carry the excess weight of the stiffening system.

Applying similar reasoning to the paper, it is true that the author makes a certain allowance for the flexibility of the truss, but he assumes that the ratio for weight reduction found in the case of the Delaware River Bridge by applying a more correct theory than that of the rigid truss, is practically constant for spans of two, three, and four times the span length of that bridge. He does not recognize the fact that the relative flexibility may and should increase with increasing weight and length of span, and that, consequently, the ratio for truss weight reduction, instead of being constant, should gradually decrease from a maximum of 100% for very short, rigid spans and approach zero value where the span length and other conditions are such as to permit of dispensing with a stiffening system.

Thus, although the author's proportions and cost estimates approach reasonable limits for spans close to the longest thus far built, they become increasingly excessive for longer spans. Incidentally, shallow stiffening trusses lend the bridge a lighter, more graceful, appearance than deep, rigid ones.

What has been said regarding the vertical stiffening system applies, with much less (yet appreciable) effect, to the horizontal stiffening system. The author properly recognizes the fact that in large suspension bridges part of the wind or lateral forces acting on the floor is transmitted to the cable, thus relieving partly the floor wind truss and permitting it to be made lighter. Here, again, the author bases the reduction on an assumption that has no relation to the weight of the cables and the dead weight suspended from them. Yet this dead weight is most effective in counteracting the wind force.

In the design for the Hudson River Bridge, with a 3 500-ft. center span, careful studies have been made to determine as closely as possible the reac-

tionary effect of the cables on the floor wind truss throughout the entire length. It was found that the floor wind forces are not only decreased, but actually reversed at the center of the span, indicating that the wind truss acts as a three-span continuous truss with an elastic support at the center, instead of as a simple span of the full length. The moment at the center is practically reduced to zero and the maximum moment is decreased to about one-eighth the moment of a simple span. The shears, likewise, are materially smaller. The material thus saved in the floor wind truss in turn effects a substantial additional saving in the cables and towers that have to carry it.

Another outcome of excessive proportioning of stiffening trusses is the author's conclusion "that there is no economy in adopting suspended side spans for suspension bridges". Suspension of the floor from the cables, with properly proportioned stiffening trusses, is the most logical arrangement. In most cases of long suspension spans, it will be found not only economically, but esthetically, superior to trestle or pier supports.

Summarizing, the speaker desires to emphasize the outstanding characteristic of the suspension bridge, which lies in the vertical and lateral stability and inherent rigidity of heavy unbraced cables or chains. It should be the principle of good and economic design to utilize that advantageous characteristic to the fullest possible extent. Although in bridge construction up to the present this principle has not been of outstanding moment, it does become of preponderant economic importance in the field of the very long spans, which the author has embraced in his paper.

FRANK W. SKINNER,* M. AM. SOC. C. E.—According to the author, this paper was prepared as a convenience for making preliminary estimates of large suspension bridges, thus furnishing a starting point for more elaborate and precise computations. As such, it is extremely valuable and above serious criticism. As the subject matter concerns application rather than methods of computation, it was necessary to select arbitrary standards of design. Only in this way can the required comparisons be fair and uniform. Although bridge engineers may differ as to details, all of them will concede that the author's diagrams furnish a uniform basis. In fact, many of the assumed elements are entirely beyond question, such as the floor systems, foundations, and stresses.

The author's results should be of great value, mainly to the expert bridge engineer, but also to many others. The time of any specialist is valuable in proportion to his ability to produce well-balanced approximations for comparison. The time required is thus much reduced by any means which enables him to differentiate between locations, combinations, comparative costs, and span lengths. As such invaluable comparisons are usually difficult, slow, and costly, this paper enables the least expert engineer, as well as the specialist, in designing a large structure or in investigating possibilities of long transportation lines, to get close approximations of cost for different locations and different considerations. This should greatly enhance the possibility of pro-

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moting larger engineering enterprises. It should stimulate the building of big bridges.

The author has been conservative and has adhered closely to standard practice in many important matters. For example, he has used only 16 000 lb. as the unit stress for steel when 18 000 lb. has been adopted by some others, notably the American Institute of Steel Construction, and has been endorsed by Special Committees of the Society. The author's 70 000 lb. for stress in the cable wires compares with the 80 000 lb. sometimes used by other designers.

The reference to the use of "Haydite" is disappointing. Most engineers would like to know more about this product,—its advantages, cost, availability, and success in bridge floors. On a long span bridge the live load is only a small fraction of the dead load so that any material reduction in the weight of the structure tends either to a direct decrease in the cost or to a proportionately large increase in carrying capacity. In very long span bridges the imperative development of adequate stability of the structure necessitates a comparatively large minimum horizontal width, thus inviting an extremely large and wide floor surface in highway and electric-car structures. This, with the solid concrete base considered essential in good practice for pavements, involves an enormous addition to the total amount of dead load from the supporting steel framework which itself is the ultimate minimum and, in the last analysis, limits the length and cost of the structure.

If, therefore, it is possible to substitute satisfactory but materially lighter floor-slabs for the standard stone concrete foundation, it would have an important bearing on the cost and dimensions of long spans. Data bearing on this subject are important for the development of advanced standards, and will be equally valuable to the designers of highway bridges and for many types of floor construction.

The author's treatment of main cables and stiffening trusses is suitable, rational, and authoritative, and is completely successful for the purposes of this investigation. His conclusions are fortified by the data of the eye-bar cable suspension bridge at Florianopolis, Brazil. His assumptions regarding stiffening trusses are eminently sound and reliable; their necessity is well demonstrated by several important large-scale examples.

The inadequate wood and iron stiffening trusses of the famous Niagara Railroad Suspension Bridge permitted deflections of more than 3 ft. under light, slow train movements, which distortions were almost wholly eliminated by the substitution of properly designed steel trusses. The 1 200-ft. highway suspension span at Niagara Falls, because it had no adequate stiffening trusses, was thrown into violent vibrations and undulations by the passage of a single horse, and was notably affected by a dog trotting across it. The multiple-span Monongahela Bridge, at Pittsburgh, Pa., had little if any stiffening truss provision and under the passage of a heavy coal or iron truck would develop a wave probably 2 ft. high that preceded and followed the live load over all the successive spans from shore to shore. Contrariwise, the enormous concentrated live loads crossing the much longer spans of the Williamsburgh and Manhattan Bridges cause no noticeable deflection because of their efficient stiffening trusses.

Besides the constructive value of its original and reliable investigations, this paper is of great practical value on account of the many actual examples, official records and summary of many previous reports and computations, and the accumulated and tabulated data of years of active practice that have been incorporated in it. Thus, it eliminates a great amount of intricate standard computations and investigations and offers the profession an authoritative epitome of research that has high economic value and convenience and, in a few hours, possesses the reader of years of work and study.

J. A. L. Waddell,* M. Am. Soc. C. E. (by letter).—In his discussion Mr. Thomson expresses the hope that the writer will withdraw the statement that the data in the paper would enable one to estimate the cost of a bridge in a few hours. Such a hope is vain! The writer was careful to stipulate the requisite conditions. The use of the paper for rapid estimating was founded, however, on the important limitation, that "full preliminary data are furnished", as in the case of the example worked out for a 3 800-ft. span to carry three kinds of traffic. Is it not evident that, if all unit prices are provided for the computation, due cognizance must have been taken of abnormal conditions affecting the building of the substructure?

Further, the claim for rapidity and accuracy was made for "any engineer of good, general experience" and not for an "inexperienced engineer", nor for a "business man." To prepare engineering estimates of importance calls for a considerable intelligence; hence the writer's diagrams are not intended to be used by tyros and roustabouts, but by thinking men who are capable of visualizing important engineering projects. If occasionally some man who is not an engineer makes a spectacle of himself by attempting to use a book or a formula that he does not understand, in order to design or estimate on bridges, is that any reason why the Engineering Profession should not be furnished with fairly accurate information for the making of preliminary estimates of cost?

Mr. Thomson asks:

"Why should the author give even an approximate formula for the cost of the foundations? Nothing in the cost of bridge work varies more than this. Many other items besides the depth affect it."

The reason for giving the "formula", or, more properly speaking, the curves, is that the information is quite accurate. If the conditions affecting the construction are unusually difficult, an allowance is to be made therefor in the unit prices for materials in place; or, if the heights of shafts or of bases vary from those assumed in the paper, the figures taken from the curves are to be modified either by direct proportion or by good judgment.

In referring to "Tower Encasement," Mr. Thomson evidently overlooked the writer's statement that "such a method of improving the esthetics of a bridge is legitimate, but, of course, by no means necessary." The steel thus encased in a shell that is not in contact with it is certainly better protected against corrosion than if it were directly exposed to the elements. Due pro-

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vision, however, would have to be made for facility of painting the enclosed metal from staging and by the use of ample electric lighting.

Again, Mr. Thomson challenges the adequacy of a 5% allowance for contingencies. Under ordinary circumstances this is enough; but with uncertain data or troublesome conditions, it should be increased to 7 or 8 per cent. If a bridge engineer allow much more than 5% for contingencies under ordinary conditions of labor, materials, and erection, he virtually confesses that he is uncertain about the correctness of his calculations. The writer has seldom had occasion to allow more than 5%; and it is rarely that the actual cost exceeds his estimate, for he tries to include every probable item of expense and not to reduce the unit prices to the lowest possible limit.

In order to obtain a conclusive practical test of the correctness of this paper and a proof of its value to those desirous of knowing the approximate cost of any proposed suspension bridge, the writer requested from O. H. Ammann, M. Am. Soc. C. E., the general dimensions, the unit prices, and the total estimated cost, excluding approaches and omitting allowances for contingencies, engineering, administration, and interest during construction, for the proposed Hudson River Bridge on the basis of using wire cables. These details are given in the following tabulation, covering the probable costs for the future completed bridge with its two full roadways, four sidewalks, and four electric-railway tracks:

HUDSON RIVER BRIDGE, FORT WASHINGTON, N. Y., TO FORT LEE, N. J.

Dimensions.—	
Main span, center to center of piers	3 500 ft.
East side span, center to center of bearings	650 "
West side span, center to center of bearings	650 "
Vertical Clearance above High Water:	
At middle of main span (mean temperature—no load)	206 "
At ends of main span	195 "
Variation between low water and high water (mean)	4 "
Elevations at Foundations of Caissons of Main Piers (Main Sea Level Elevation = 0):	
New York side	+ 5
New Jersey side (average)	— 60
Elevations at Bottoms of Shafts of Main Piers (Main Sea Level Elevation = 0):	
New York side	+ 15.0
New Jersey side	+ 15.0
Width of upper roadway in the clear, 2 by 24 + 40 ft.	88 ft.
Width of lower roadway in the clear	None
Total clear width for sidewalks, two at 6 ft.	12 ft.
Total width for railways, four tracks (open floor)	40 "
Unit Prices.—	
Silicon steel, in place	$0.09 per lb.
Nickel steel, in place	0.11 " "
Carbon steel, in place	0.08 " "
Wire cable (wrapped), in place	0.20 " "

Pier shafts, inclusive of steel, $80 per cu. yd.	$30.00	per cu. yd.
Pier bases	20.00	" " "
Anchorages	25.00	" " "
Cost of base for pavement, including reinforcing metal	7.25	per sq. yd.
Cost of pavement (2-in. asphalt)	2.75	" " "
Cost of sidewalks, including reinforcing metal	5.00	" " "

Total Cost of Main Bridge, Including Anchorages.—

New York main pier, complete, tower and foundation	$6 770 000
New Jersey main pier, complete " " "	7 660 000
New York anchorage, complete	4 540 000
New Jersey anchorage, complete	1 500 000
Highway floors, with pavement, in main span and side span (exclusive of steel I-beams)	470 000
Sidewalk floors in same	35 000
Railway floors in same	135 000*
Metal, excluding cables, in main span (excluding towers)	5 820 000
Metal, excluding cables, in two side spans	2 000 000
Wire cables, complete, in main span	9 400 000
Wire cables, complete in side spans	4 500 000
Lighting system for main span and side spans	280 000
Total cost	$43 110 000

Using Mr. Ammann's data, the writer prepared Table 5. As costs are more likely to over-run than to under-run the estimates, the comparatively small variation, 4.2% on the side of safety, is by no means objectionable.

In Table 5 four necessary and self-evident modifications have been made in the values taken from the diagrams, namely, those for pier shafts and pier bases. Mr. Ammann's vertical clearances are 60 ft. (or 50%) greater than those assumed for the paper, hence the readings taken from Fig. 11 have been multiplied by 1.6, and those for the bases (on this account) by 1.1. There are two other factors for the base readings, namely, 0.5 because only one of the two main piers has an under-water base, and 0.6 because the average depth of that base is 60 ft. instead of 100 ft. These modifications are unequivocally self-evident.

Mr. Muckleston mentions the porosity of Haydite. As far as the writer has been able to learn, it is less porous than ordinary concrete. Mr. Ammann is now conducting an elaborate series of tests on it, and will soon be in a position to speak authoritatively on the question of the suitability of Haydite for bridge floors. Thus far, unfortunately, the strength of the 1:2:3 mixture at 28 days has fallen considerably below that of ordinary concrete, but some tests with the addition of celite, to be made later, may give better results.

Mr. Muckleston's suggested type of timber bridge floor can, with proper protection, be made almost entirely safe against fire, hence it is a legitimate construction for long-span suspension bridges in which a low first cost is an important desideratum.

His scheme for reducing the cost of anchorages by building reinforced concrete boxes and filling them with sand or gravel is feasible and legitimate; and at first thought it appears to be economical, but actual competitive figures

* This minor item was supplied by the writer.

are necessary to determine what saving can be effected by the innovation. The writer agrees that steel towers are preferable to masonry ones, and that tower encasement is not necessary.

TABLE 5.—Comparative Estimate for Hudson River Bridge by the Method of the Paper.

Item.	Quantity per square foot of roadway.	Unit price.	Product.	Equivalent area, in square feet per linear foot.	Cost per linear foot of span.
Highway slab	0.02 cu. yd.	$33.00	$0.66	88	$58.08
Pavement	0.11 sq. yd.	2.75	0.33	88	29.04
Sidewalk slabs	0.012 cu. yd.	45.00	0.54	12	6.48
Railway floor			0.70	40	28.00
Floor system, highway (Fig. 3)	48 lb.	0.08	3.84	94	360.96
Floor system, railway (Fig. 3)	49 lb.	0.08	3.92	40	156.80
Lateral system, average of highway and railway (Fig. 4)	15.5 lb.	0.08	1.24	134	166.16
Stiffening trusses, highway (Fig. 5)	65 lb.	0.11	7.15	94	672.10
Stiffening trusses, railway (Fig. 5)	100 lb.	0.11	11.00	40	440.00
Suspenders, average of highway and railway (Fig. 7)	7 lb.	0.11	0.77	134	103.18
Main cables, highway (Fig. 8)	102 lb.	0.20	20.40	94	1 917.60
Main cables, railway (Fig. 8)	132 lb.	0.20	26.40	40	1 056.00
Back-stays, highway (Fig. 9)	82 lb.	0.20	16.40	94	1 541.60
Back-stays, railway (Fig. 9)	110 lb.	0.20	22.00	40	880.00
Towers, highway (Fig. 10)	75 lb.	0.09	6.75	94	634.50
Towers, railway (Fig. 10)	103 lb.	0.09	9.27	40	370.80
Tower encasement (Fig. 13)			4.25	134	569.50
Lighting			0.20	140	28.00
Pier shafts, highway (Fig. 11)	$1.6 \times 0.147 = 0.235$ cu. yd.	30.00	7.05	94	662.70
Pier shafts, railway (Fig. 11)	$1.6 \times 0.157 = 0.251$ cu. yd.	30.00	7.53	40	301.20
Pier bases, highway (Fig. 12)	$\frac{1}{2} \times 0.6 \times 1.1 \times 0.155 = 0.051$ cu. yd.	20.00	1.02	94	95.88
Pier bases, railway (Fig. 12)	$\frac{1}{2} \times 0.6 \times 1.1 \times 0.168 = 0.055$ cu. yd.	20.00	1.10	40	44.00
Anchorages, highway (Fig. 14)			5.60	94	526.40
Anchorages, railway (Fig. 14)			8.00	40	320.00

Total cost per linear foot of bridge proper on the assumption of there being no side spans =	$10 968.98
Total cost on that basis = $10 968.98 × 3 500 =	$38 391 430.00
Cost per linear foot of superstructure for side spans between towers taken from Table 5 = $5 022.40	
Total cost of superstructure for side spans = $5 022.40 × 1 300 =	6 529 120.00
Total cost of main structure =	$44 920 550.00
Engineer's total cost =	43 110 000.00
Difference =	$1 810 55 .00

Percentage of difference = $\frac{\$1\ 810\ 550 \times 100}{\$43\ 110\ 000} = 4.2$ (excess) by the use of the diagrams.

Mr. Dana's remarks on the elastic theory and the deflection theory are very clear and convincing; and his statement that for exceedingly long spans the writer's weights of metal in stiffening trusses are much too great is correct. When the paper was being prepared, the writer recognized that his truss weights for very long spans were greater than those that might be found by the use of the deflection theory; but he did not anticipate how large the excess would be. Mr. Ammann's subsequent elaborate stress computations

indicate that, for extremely long spans, the stiffening trusses are almost unnecessary.

Mr. Dana's discussion of the omission of overhead bracing is very interesting. If actual measurements of chord stresses in finished bridges under load confirm his theoretical conclusions, the writer will be ready to concede that such bracing is not necessary, especially for long spans.

Mr. Oesterblom's statements about the suitability of Haydite concrete for bridge floors are very gratifying. For many years the writer has been urging the inventor, Mr. Steven J. Hayde, of Kansas City, Mo., to have suitable experiments made, in order to prove its fitness.

Mr. Moisseiff questions the value of the paper because of the small demand for long-span suspension bridges; but if any such structures are to be built in the future, the information furnished would be of value to engineers. The discussions of the paper certainly provide much information of value to all bridge engineers—including Mr. Moisseiff's own comments—justifying the writer's efforts. Mr. Moisseiff is correct in his surmise that the paper was not intended to furnish rules for design, but simply to enable one quickly to make preliminary approximate estimates of cost. There is still a great deal to be learned about the theory of design for long-span suspension bridges; but much knowledge is now being accumulated by the bridge specialists who are calculating such structures.

Answering Mr. Eberly, the reason that the writer did not consider short-span suspension bridges is that his past investigations have shown that such bridges are not economical for spans much less than 1 000 ft. in length as compared with cantilever structures. All the curves of the paper were carried by extrapolation down to spans of 500 ft. In the writer's opinion, that is certainly short enough, even for bridges of the new self-suspending type, as at Pittsburgh, Pa.

Mr. Van Cleve's discussion of the rising costs of bridge materials and labor in this country during the last twenty years should prove useful to the Engineering Profession.

Answering Mr. Brown's comment on the cost of lighting, the minimum for effective service is 10 cents per sq. ft. of floor and double that amount will provide quite an elaborate lighting system, but it is feasible to spend 30 or 40 cents per sq. ft., if one desires real decoration by illumination. As Mr. Brown points out, the comparative cost of lighting is so small that it would have no material effect on the grand total cost of the structure.

Mr. Grove's discussion of the subject of eye-bar cables *versus* wire cables is enlightening; but the only real test of value would be actual competition by bidders on several proposed suspension bridges.

Answering M. Godard's questions, the writer would state that unit stresses for steel have been determined by long experience and by the work of numerous committees of such societies as the American Society of Civil Engineers, the American Railway Engineering Association, and the American Society for Testing Materials. This work has extended over many years. When improvements have been made in the manufacture of the metal, the unit

stresses have been raised accordingly. Again, the more exactly the actual stresses in bridge members have been determined, and the more knowledge that has been accumulated concerning the effects of impact, the more confidently have bridge engineers modified practice in relation to intensities of working stresses.

Of late unit stresses for steel wire have been gradually increasing, because of the greater elastic limits and ultimate strengths that are being attained by manufacturers. Owing to the scientific and thorough manner in which wires are being strung in America, the stresses that come upon the cables must be very uniformly distributed. Consequently, the assumption that only 80% of their theoretical capacity can be relied on would be unnecessarily extravagant of material. Certainly 90% can be counted on, possibly considerably more. One eminent authority has stated to the writer that he allows as much as 95 per cent.

There is a curious thing about wire cables, namely, that the splicing of the wires does not seem to reduce the strength of the cable. This has been shown by reliable tests.

The writer's method of determining the working unit stress for wire cables was as follows:

Ultimate strength of individual wires = 240 000 lb. per sq. in.
Coefficient for group of wires = 0.9.
Factor of safety for ultimate strength = 3

$$\text{Unit working stress} = \frac{240\ 000 \times 0.9}{3} = 72\ 000 \text{ lb.}$$

For conservatism this stress was taken at 70 000 lb.

The latest studies made on long-span, wide-floored suspension bridges indicate that it is perfectly legitimate to count on a very small wind load actually reaching the horizontal truss or trusses composing the lateral system. One reason for this is that the longer the span the less will be the actual average wind load per square foot of exposed surface. Another is that, when the cables are blown out of line, a large part of the wind load is exhausted in maintaining them there. Finally, another large part is transported to the piers by the great mass of the slab flooring and its pavement—and this in spite of the provision of numerous expansion joints.

Mr. Ammann's discussion adds materially to the general knowledge concerning suspension bridge designing. He differs from the writer about the necessity for over-head wind bracing on very long spans; and, in view of his wider experience on suspension-bridge work, the writer concedes the point and probably will design his future suspension bridges accordingly. The same remark applies to Mr. Ammann's criticism concerning the proportion of the wind load that is carried by the lateral system.

As for the reduction in the strength and rigidity of stiffening trusses, in view of the several failures of members in those of the old Brooklyn Bridge, the writer would be inclined to go slowly. He recognizes, of course, that such failures could not involve any disaster or even serious inconvenience; because the continued existence of a suspension bridge is not at all dependent on the integrity of its stiffening trusses.

The writer heartily thanks Mr. Skinner for his appreciative discussion of the paper. He has thoroughly comprehended its intent and purpose; and his endorsement of its aims and objects should certainly aid materially in promoting its usefulness to the Engineering Profession.

The writer's partial adherence in the paper to his old ideas concerning suspension-bridge designing involves some errors on the side of safety in respect to total costs of structures; but, as shown previously, the summation of these errors, in the case of Table 5, amounts to only 4.2 per cent. For longer spans than 3 500 ft. the error would be greater, and for shorter spans it would be less. Possibly for spans of minimum length it would reduce to zero or even reverse a little. As the sole object of the paper is to provide a means of making quick, fairly accurate, and perfectly safe estimates of cost, there is no reason for correcting any of the curves of quantities of materials in order to have them agree more closely with the latest methods of making suspension-bridge computations.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1650

THE PANAMA CANAL

ADDRESS AT THE ANNUAL CONVENTION AT DENVER, COLORADO, JULY 13, 1927

By John F. Stevens,* President, Am. Soc. C. E.

The epic story of Jason and his fellow Argonauts, which describes their quest of the Golden Fleece, contains nothing more of romance than does the recital of the adventures of those bold spirits whose daring, skill, and hardihood, so ably carried on the work of Columbus in giving to the world the continents of the Western Hemisphere.

Columbus, skirting the west shore of the Caribbean Sea in his futile search for a short passage to Cathay with its fabled riches of gold, silver, and precious stones, visited the shores of Panama on his fourth and last voyage in 1502. He named the harbor of Porto Bello and examined Limon Bay, which bay is now the Atlantic entrance to the Panama Canal.

All history points to the conclusion that Columbus died in the belief that the lands which he had discovered were a part of the Continent of Asia. The cosmography of those faraway days was incomplete and misleading, but it was shortly to be extended, modified, and corrected by a group of adventurers, the story of whose exploits would furnish material for another immortal Odyssey.

Magellan, the great navigator, discovered the straits which bear his name in 1520, and at that time he named the vast body of water to the westward the Pacific Ocean, by reason of its calm and peaceful aspect as he saw it. Sir Francis Drake, probably England's most picturesque and most unscrupulous adventurer of all time, with the exception of Henry Morgan, visited the Isthmus in 1570, captured the Spanish town of Nombre de Dios and burned Porto Bello. Crossing the Isthmus to a high point he saw the waters of the great ocean to the westward. Fifty-seven years before Drake, however, Balboa had crossed the Isthmus, somewhat southeast of the line of the present canal,

* New York, N. Y.

and undoubtedly was the first white man to view the Pacific Ocean. Balboa later fell a victim to the jealousy of the Governor of Panama and was beheaded by him.

The hope of finding a hidden strait connecting the two great oceans, persisted long years after Balboa's discovery, and it is a most interesting fact of history, that it was four hundred years, almost to a day, September 26, 1913, when such a strait was given to the world, but it was not a strait made by Nature, but by man—the Panama Canal.

Sir Francis Drake circumnavigated the globe in 1578 and 1579, probably touched our western coast near the Golden Gate, and returned to England by way of the Cape of Good Hope, the southern point of Africa. Following him were other historical navigators and the result of all their voyages confirmed as a fact, that only a narrow neck of land, fifty miles in width, separated the two oceans at what is now known as the Isthmus of Panama. Morgan, in 1671, crossed the Isthmus with a small force and after enduring terrific hardships captured, sacked, and burned the Spanish town of Panama, and butchered many of its inhabitants. The remainder after living in the jungles for a year or more, built the nucleus of what is now the present City of Panama, a city of about 50 000 or 60 000 people. Morgan used his ill-gotten wealth so wisely that he became Sir Henry and died in his bed an English Christian Knight. The Spaniards, with cross in one hand and sword in the other, proclaimed that their purpose was to carry their religious faith to heretic people, but they were far more actuated in their operations by their thirst for the riches supposed to exist in those unknown lands. By enforced native labor they built a stone-paved pack trail about 2 m. in width across the Isthmus from the Atlantic to the Gulf of Panama, over which was carried the tons of silver which Pizarro and his fellow bandits, the conquerors of Peru, wrested from the hapless people of that unfortunate country. Remnants of this old trail still existed in 1905 and it was my pleasure to ride over some of them in that year. The Panama Railroad was not built until about 1852 and then as the result of the rush of travel consequent upon the discovery of gold in California in 1849.

As nothing in the nature of a strait could be found north of Magellan, which would allow of the passage of ships from ocean to ocean, the persistent leaders began to talk of making one. Saavedra, who accompanied Balboa on the latter's visit to the Isthmus which resulted in the discovery of the Pacific Ocean, is said to have been the first to prepare plans for a canal, which it is claimed he did in 1529.

From that time on, through centuries, several nations—Spain, France, Britain, and America—made surveys and developed projects. Quoting history, "it made an inescapable appeal to popular, as well as to commercial and naval imagination." Nothing tangible or conclusive beyond surveys and discussions came out of it all, until the ill-fated de Lesseps' project was launched with loud acclamations in 1882. Fresh from the successful completion of the Suez Canal, and enjoying the prestige of its accomplishment, then of its nature the greatest in history, Count de Lesseps, operating under a concession of the then

proprietary Government of Colombia, undertook the digging of a sea-level canal through the Isthmus of Panama. He soon discovered that such an undertaking was vastly different from the one he had carried out at Suez.

The history of the failure of the French is of a time too recent and too fresh to need repetition here. As the inability to raise the funds needed for construction, owing to difficulties unforeseen and unprovided for, became more and more pronounced, this history, as I have studied it, was amply illustrated by the forced changes in plans in the famed Culebra Cut; first a change to a one-lift high-level canal, then to more locks and higher levels, all of which was plainly evident on the ground, and the contemplation of these changes was really pathetic. But with an ample supply of funds, their attempt to build any sort of a canal there would have failed, as undoubtedly would the Americans have failed, until that then scourge of the white man in the tropics—yellow fever—had been mastered.

The de Lesseps Company was re-organized in October, 1894, and continued operations on a very limited scale, probably to enable it to hold its concession, until its rights and property were purchased by the United States under the authority of the Act of Congress of June 28, 1902. As soon thereafter as conditions and events permitted, active work of construction of a canal was begun by our Government, and pushed to a successful conclusion. Some of the interesting features of the initiating and executing of this gigantic enterprise have never been properly set forth, and it is my purpose here, to make them known without prejudice, conceiving that, as far as time will permit, a plain narration made by one who was intimately connected with the work during its earlier and formative stages, can only redound to the credit of the United States and to the vision of its engineers.

Under the authority of Congress, the President, in 1899, appointed a body of men known as the Isthmian Canal Commission, with orders to "investigate any and all practicable routes for a canal across the Isthmus of Panama, and particularly to investigate the two routes known, respectively, as the Nicaraguan route and the Panama route, with a view of determining the most practicable and feasible place for such canal." This Commission pursued the work of investigation as instructed, until as stated, the matter was closed by the purchase by our Government of the de Lesseps' rights and property at Panama. Then began the efforts of the Commission to round affairs up in shape so that plans could be made and the actual work of construction be gotten under way.

But by 1904, the Commission had amply demonstrated its inability to outline and to carry on any intelligent or progressive program of advancing the work, or of offering any hope for the completion of any kind of a canal anywhere, in any reasonable length of time, so its members were relieved and a new commission was appointed by the President.

The late John F. Wallace, Past-President, Am. Soc. C. E., had been the Chief Engineer for a year or more under the old Commission, and remained as such under the new Commission, until the early months of 1905, when he resigned the office and left the Isthmus. Mr. Wallace was a man whom I had

known for years, and esteemed, not only as a warm personal friend, but also for his great ability as an engineer. The statement is made here, that his action in resigning, under the conditions and the handicaps of the old Commission, was perfectly justifiable. His zeal had been dulled, and as he expressed the situation to me, he had no hopes that the new Commission would prove any more effective than the old one.

No one who knew him could justly impugn his high character or his ability as an engineer, and if any comment could fairly be made as to his conduct, or of his action in resigning, it would be that a more strenuous attitude toward his superior officers would have been more efficacious and would have tended toward a more active state of affairs. Mr. Wallace was not an aggressive man, and there are times and conditions when fighting becomes a righteous duty. The situation which had developed at the date of his resignation, was such as demanded a one-man control, control by a man who was heedless of his technical reputation; and one only bent upon smashing a way through all obstacles, a kind of politic "roughneck", who did not possess too deep a veneration for the vagaries of constituted authority.

Whether I filled the above specifications or not, against my own inclination, I was appointed to the office of Chief Engineer, succeeding Mr. Wallace. My acceptance of the position was brought about by the persuasive tongue of William Nelson Cromwell, whose reputed good work secured the possession, by the United States, of the French rights and property on the Isthmus. It was plainly intimated to me that the Administration was gravely concerned as to what the reaction of the American public would be if another failure to build the canal took place, and what the effect would be upon its own fortunes; so a final refusal to accept the position became impossible, and accordingly such appointment was made.

Previous to my departure for the scene of my future labors as the new Chief Engineer, I had a conference with President Roosevelt at Oyster Bay, on which occasion I laid down certain conditions, as nearly as conditions can be made to the President of the United States. One of these was that, while technically I would be under the orders of the Commission, I was to have a free hand in all matters, and would be hampered by no one in authority, high or low. Another one was, that I would not agree to remain until the completion of the canal, but would remain until the failure or success of the undertaking was assured according to my own judgment, and I fulfilled my promise in regard to the latter result, to the very letter.

The President made no objections to these statements, simply saying that the Canal must be built, and that he depended upon the new Chief Engineer to make it a success. In no way whatever, at any time, did he give orders as to how matters should be handled. He made some suggestions, some of which were good, and some which were not adopted. He gave one intimation at the conference mentioned, and gave it in his characteristic manner, by an anecdote. A certain man suddenly became wealthy and set up a large establishment as a home. When his butler arrived he said to him, "I don't know in the least what you are to do—but one thing I *do* know, you get busy and buttle

like Hell!" He made some comments upon the old Commission which it is not necessary to quote here. He said furthermore that he did not know how affairs were, down on the Isthmus, but he thought they were in a "devil of a mess". When I reached the Isthmus, I found that the President was correct to a superlative degree in his opinion.

The President advised that I should address him directly in regard to matters connected with the work that I was about to undertake, and when the point was made, that such action might raise conflicts with the Secretary of War, or with the Commission, he brushed the objection aside, saying that all parties involved understood his wishes in the matter. As a consequence these wishes were carried out to a remarkable extent as far as information was concerned.

All reports that were current during the early period when the staff on the Isthmus was being organized, as to political influence in the selection of its personnel were false, as far as I was concerned. No such attempts were made, no officer of the Government and no member of Congress sought such favors for relatives or friends, either directly or indirectly. I was entirely free to choose my subordinates according to my judgment of their fitness.

An attempt was made by the head of the American Federation of Labor to intimidate me into granting special conditions for a certain kind of Union labor, in fact, to make a closed shop; and when such demand was rather forcibly refused, he threatened to take the matter to the President, and was told that he could take it to the Lord, if he chose, but that it would make no difference, and to close the door when he went out, which he did at once, and nothing further was heard on that subject.

The marine equipment of the Panama Railroad which had been taken over by the Government, included five small steamships in service between New York and Colon. These ships had just been fitted with refrigerator space and wireless outfits, the latter then only just coming into use. As practically all food supplies needed for the white employees on the Isthmus must come from the United States, a contract was made with a Chicago packing house for a regular supply of fresh meats, the prices for the same to be on a sliding scale, to be regulated by the wholesale prices as posted daily in Chicago. It may be said here, that such an arrangement proved very satisfactory in providing a regular supply of the best quality of everything, and at prices to the Isthmian employees at about 60% of the retail prices for the same meats right in Chicago.

After seeing that such necessary matters were properly attended to, I sailed for the Isthmus in company with the Chairman of the Commission and others, the Chairman remaining there only a few days, as health and other conditions did not seem to appeal strongly to him.

The first few days after arrival were devoted to getting as close a view of the situation as possible. It did not take long to realize that the discouraging reports current in the United States had not been overdrawn, but rather understated. There was no real organization, no competent directing head of affairs, few laborers, and what small amount of work was in progress, was in

most cases merely colorable and of little permanent value; but one outstanding fact was plainly evident and that was, that the problem of technical engineering must be relegated to the background until the fundamentals had been established; until something like a working organization had been created; until intelligent and comprehensive plans had been made and put into execution to collect a substantial working force; and, above all, until there had been established a proper morale and a belief that the Canal could and would be built, something that was lacking. The contrary belief was not wholly confined to the Isthmus, as before leaving the United States, otherwise intelligent and well-meaning friends encouraged the new Chief Engineer, by telling him how sorry they were that he had been picked as the "goat," as no one could build a canal at Panama. An optimist at that time was a rarity.

The great overshadowing cloud which hung over the enterprise, was the need of sanitation, and the crux of its needs was the all-compelling, absolutely essential requirement of clearing the Isthmus of yellow fever and of keeping it clear. While the situation was in a measure psychologic, there was a real grave danger which could not be ignored, and which would have imperiled the very life of the undertaking, had it not been removed. Whatever could be done to change the situation and restore confidence, must be done at once, done effectually regardless of method, cost, or personalities.

One of the first men that I had the good fortune to meet there, was Colonel (afterward General) Gorgas, the Chief Sanitary Officer, who at that time was reporting to the Governor of the Canal Zone. Literally, he was being supported in his tremendously important and vital work by no one. Col. Gorgas was one of the finest characters that I ever knew, and it is a source of gratification to remember the friendship which resulted from close association with him, something more than official.

Like probably many others I had gained some little idea of the mosquito theory of the dissemination of yellow fever, by reading of the experiments of Italian scientists on the Roman Campagna, and had known something of the work of Walter Reed, Col. Gorgas, and others in Cuba, but, like most laymen, I had little faith in its effectiveness, nor even dreamed of its tremendous importance. But as should always be the case in such matters, an open mind was held on the unfamiliar subject, and it was well that the matter was so approached, for it seemed simply an act of Providence that just at that time such men as Gorgas and his very able assistants were on the job.

Several conferences were at once held with Col. Gorgas, who patiently and fully explained conditions, and what could be done to mitigate them, if only he could be given the means and support to do so. It was plainly evident from observation, that he had an efficient, loyal, and enthusiastic staff. Also, that he had very scant hospital space, practically no field working force commensurate with his needs, and that a greater part of his forces, skilled and unskilled, were quartered in makeshift houses mostly those of the old French régime. That, worst of all, a policy, not only of neglect but of absolute indifference and derision was openly expressed as to the value of the mosquito

theory, and it was being carried on by his superior officers, on whom he had relied for support.

I myself heard expressions from these officers, such as how "that crank" was wasting money in trying to drain water pools, in cutting grass and weeds, and insisting that every house and quarter should be screened against mosquitoes. Finally, the Chairman of the Commission advised me, that he had decided to eliminate Col. Gorgas—"to fire him", as he elegantly expressed it—and that he proposed to put a young osteopathist whom he had brought with him from the States, in Gorgas' place. My reply to this plan was more forcible than elegant, and perhaps would not bear repeating here and if paraphrased would lose most of its picturesque punch, but it had the merit of being effective, and Gorgas remained to carry on his wonderful work until the completion of the Canal. He and his staff were the men for the place, but if he and they had not been given the support when and how it was, as I shall detail later, it is likely that his work would have gone for naught, with more than a probability that another dismal failure would have been recorded and we would have had no Panama Canal to-day.

As related by Col. Gorgas, he had received little or no support from either Commission, the Chairman of the former and some of its members openly deriding his efforts, but his courage was an inspiration, and he thoroughly converted me to what was the truth—that he was on the right track to make a success.

So casting aside all question of existing authority or matters of official etiquette, cutting red tape entirely, I told him that the whole weight and resources of the Engineering Department—men, material, and money—were at his command. Laborers at this time were pitifully few in number, but they were turned over to the Sanitary Department as fast as called for, and other work, however important it seemed to be, was made to wait. The work of building houses and other living quarters, additional hospital accommodations, and of providing all classes of structures occupied by human beings with modern sanitary fixtures and screens, was pushed as fast as material and men to handle it could be obtained.

In a comparatively short time the good results of the work of the Sanitary Department became so apparent that most of its critics were stilled, even if some of them were not converted and failed to share the enthusiasm of the Chief Engineer. But it should never be forgotten, that the Engineering Department played a great and very vital part in the sanitation of the Isthmus, which made the success of the Canal possible. It was shortly after this time when Gorgas, at the suggestion of the Chief Engineer, was made a member of the Isthmian Canal Commission.

An interesting incident occurred some years after my connection with the work had ceased. President Roosevelt had been succeeded by President Taft, and the former had his office with the Outlook Company in New York. One day word was brought to me, that Col. Roosevelt wished to see me, consequently a call was made upon him.

After some pleasant remarks, Col. Roosevelt said that the trouble between Col. Gorgas and Gen. Goethals had evidently reached a climax, and that one or the other would have to go, and likely that one would be Gorgas. He said that he (Roosevelt) was well aware that I knew as much or more of Gorgas and his work than any man, and asked if he was the man for the place, and if he should be kept. I knew little of the merits of the controversy, but I did know Gorgas. I could barely find words to express my appreciation of Gorgas and his efficient work, and what the Sanitary Department under Gorgas had done, to enable me to win out in the face of what had appeared to be insurmountable difficulties, and in closing, I said that "if every other official on the Isthmus were called off, Gorgas should remain, and that his removal would be a stupid blunder."

Col. Roosevelt, pounding on the desk in his usual vehement manner, exclaimed, "By George, that settles it, Gorgas stays." And stay he did, until the completion of the Canal.

It has been a source of wonderment since that occasion, bearing in mind the not altogether harmonious relations which were reported to exist at that period between President Taft and Col. Roosevelt, how the latter had the influence, or power it may be called, to keep Gorgas where he belonged, at the head of the Sanitary Department of the Canal Zone. It was fortunate that he had such power, if he used it.

Below is quoted verbatim, a letter received from Gen. Gorgas, shortly after his appointment as Surgeon General of the Army:

"WAR DEPARTMENT
OFFICE OF THE SURGEON GENERAL
WASHINGTON

"APRIL 17, 1914.

"MR. JOHN F. STEVENS
875 Park Avenue
New York City

"DEAR MR. STEVENS:

"Yours of March 8th is acknowledged. I have a very clear and grateful recollection of the support and friendship which you always gave me on the Isthmus. I know very well at the time you refer to that you were the only one of the chief officials who believed in the sanitary work we were doing, and who was not taking active measures to oppose us. The fact is that you are the only one of the higher officials on the Isthmus who always supported the Sanitary Department, and I mean this to apply to the whole ten years, before and after your time. So you can understand that our relations, yours and mine, stand out in my memory of the trying ten years I spent on the Isthmus, as a green and pleasant oasis.

"I am glad your health is good and you are all right. I have just gotten settled in my new work here in Washington and look forward to the coming years with pleasure.

"Mrs. Gorgas joins me in kindest regards to yourself and Mrs. Stevens.

"Sincerely yours,
"W. C. GORGAS."

Needless to say that I treasure this letter as one of my most cherished papers. Considerable space has been given here to the sanitation of the Isthmus, without which the Canal could not have been built, so that the members of the American Society of Civil Engineers may realize that something other than technical engineering was done there during 1905 and 1906, by members of this great Society.

With sanitation assured and a well grounded faith that yellow fever would soon become a thing of the past, the way was clear to form an organization commensurate with the importance of the project. Enough details of such an organization had to be provided to care for the conduct of the work for a period of years, no matter what type for the proposed Canal was chosen. No time was lost in waiting for the results of sanitation to become apparent, for within a short time after reaching the Isthmus the Chief Engineer felt well assured that success was fully made certain. There was already in service first-class material sufficient to form the nucleus of an organization. All it needed was faith in a leader, whom it could trust and follow with confidence.

The various classes of the work, as far as then could be seen, were segregated, and selection was made of what appeared to be the best men to head each department, and almost without exception the selections proved to have been wisely made. One general policy was adopted and strictly followed, and that policy faithfully pursued is the only one that leads to efficient administration, it is to give an official ample authority, and then to hold him responsible for results. A maxim laid down, and which soon was thoroughly understood, was that if a man was right three times out of four, he was a success, but that if he wanted to make his failure certain with the Chief Engineer, he should do nothing of his own initiative, but "pass the buck".

The personnel was strengthened from time to time by additions to its members, and it was not long before an organization was built up, sufficiently strong and yet flexible enough to meet all demands. Considering the magnitude of the undertaking, its uncertainty at first, the condition into which it had been allowed to drift, and its location in a tropical country, the statement is justifiable that never in the history of construction projects was any organization better fitted for its purpose.

The lack of a decision as to what type of a canal should be built, was not so much of a handicap as might be supposed. Specifications were made, and plant material and supplies were ordered in vast amounts and of endless variety. And it was very satisfactory later to know that not a thing was gotten but what was well adapted for, and was used in the progress of, the work. And much of such work was in progress and the greater part of the plant provided, before the type of the canal was finally decided upon.

The discussions began as to whether a so-called sea-level (a misnomer by the way) or a lock type of canal should be adopted. The alluring phrase "sea level" naturally caught the fancy of the multitude, and after a majority of the International Board of Engineers, which President Roosevelt had convoked, had reported in favor of such a type, the matter seemed to be settled. But some of our most competent engineers who had given the matter close

study, were of the opinion that the lock type, considering all the factors which entered into the problem, was eminently the proper solution, and the key to the proposition was the Gatun Dam.

This dam had been suggested some years previously and it appealed to the Chief Engineer as a certain means of curbing the unruly Chagres River and making it the servant, instead of the master, of the situation, and he became an ardent advocate of the lock type, such as was ultimately selected, and which is proving so successful in operation.

The President had an open mind as to the choice of type, probably he rather inclined to the sea level. The Chief Engineer had several conferences with him and with the Secretary of War, and argued to the limit of his ability in favor of the lock type, which the President recommended to Congress as the proper type to be selected.

In a report as of date January 26, 1906, which the Chief Engineer made to the Isthmian Canal Commission, the following language was used:

"The sum of my conclusions is, therefore, that, all things considered, the lock or high level canal is preferable to the sea level type, so called, for the following reasons:

"It will provide as safe and a quicker passage for ships, and therefore will be of greater capacity. It will provide beyond question, the best solution of the vital problem of how safely to care for the flood waters of the Chagres and other streams. Provision is made for enlarging its capacity to almost any extent, at very much less expense of time and money, than can be provided by any sea level plan. Its cost of maintenance and fixed charges will be very much less than any sea-level canal. The time and cost of its construction will not be more than one-half that of a canal of the sea level type. The element of time might become, in case of war, actual or threatened, one of such importance that measured not by years but by months and even days, the entire cost of the canal would seem trivial in comparison. Finally, even at the same cost in time and money for each type, I would favor the adoption of the high level lock canal plan, in preference to that of the proposed sea level canal. I therefore recommend the adoption of the plan for an 85-foot summit level lock canal, as set forth in the minority report of the Consulting Board of Engineers.

"Very respectfully,

"JOHN F. STEVENS,
Chief Engineer."

The report of the Commission to the Secretary of War, and the latter's report to the President, embodied these points, and the President in his message to Congress recommended the adoption of the lock type. His recommendation was adopted by Congress, quickly by the House, but only after some delay and discussion in the Senate, by the small majority of four votes, small, but enough. The able speech of Senator Knox on the floor of the Senate, in favor of the lock type, was probably the deciding factor, but without violating any confidence it may be said here, that much of the speech was read directly from type-written sheets dictated by the Chief Engineer.

Soon after the type of the proposed canal had been decided, the question arose as to how, that is, by whom should the work be carried out, directly by Commission forces or by contractors. It was rather felt that strong contractors who had trained organizations and large personal followings, could better

handle such enormous work than could a new and untried organization, and that it would be better not to have the Government go into such business directly, and the Chief Engineer was at first rather inclined to sympathize with that belief. Accordingly, he was asked by the Commission to submit a form of contract for its consideration, which he did.

Owing to the great multiplicity of details involved, it was manifestly impossible to ask for bids upon unit quantities, so, using as a basis a modified form of percentage contract which he had successfully employed in heavy railroad construction work, and which had been passed upon by competent legal authority as legally air-tight, he prepared a form, modified and hedged about in such a manner as to make the Chief Engineer supreme in all matters and questions that might arise, for the chances of such were numberless.

The discussion of this proposed form—of asking for tenders—was taken up directly by the President at a Cabinet meeting at which the Chief Engineer was present. It met the tentative approval of the President and members of the Cabinet excepting that of Secretary Root, who insisted upon injecting some conditions which practically destroyed its value, but of course his views prevailed, and the Chief Engineer left for the Isthmus knowing full well what the result would be.

Tenders were invited and received from several syndicates, and a summary of the bid which was considered most favorable, was cabled to the Chief Engineer, who, after due deliberation, cabled his disapproval, with a recommendation that the Commission carry out the work directly itself through its Engineering Department. This report was approved by the President, and all idea of letting a contract was dismissed.

The reasons which caused the disapproval of the bid by the Chief Engineer were several in number. One was that, in view of the magnitude of the enterprise, the percentages asked were too large. Another reason was that the contract form had been so emasculated by the introduction of Secretary Root's changes, as to make it practically impossible to carry out the work without endless friction and conflicts of authority. Another reason, and not the least, was that the Chief Engineer had no confidence in the administrative ability of the parties with whom he would have to deal if the contract was awarded to them. And so the work was carried through by the engineers, and its execution, both from a technical and business standpoint, forms one of the brightest pages in the history of construction projects.

It will be noted that the sea-level type has been designated here as, "so-called" or "proposed". In explanation, please note that no operating ship canal at the level of the oceans could function, without the introduction somewhere in its length, supposedly near the Pacific entrance, of a tidal lock, unless the canal was of a much wider section than was recommended by the majority of the Consulting Board of Engineers. The difference of the tides at the Atlantic and Pacific ends is generally 20 ft. and often more than that.

As recommended by the majority of the Consulting Board, the sea-level canal for 21 miles would have had a bottom width of but 150 ft., without turnouts, and at the depth to which it would have been dredged, with rough,

rocky sides—a narrow, tortuous, crooked waterway, with thirty torrential side streams pouring down precipitous slopes into its section, in a country where 1½ in. of rain per hour has been recorded for 10 consecutive hours. No speed that would permit of steerage way could have been maintained under such circumstances in such a ditch, and the slightest impact of a large ship against the rocky sides, would have crushed her plates like eggshells. Competent, experienced shipmasters have expressed themselves that no sane navigator would have risked his ship in such a channel.

It was proposed to hold back the Chagres River by a dam 180 ft. high to be built at Gamboa, directly alongside the Canal, and the overflow from the river was to be taken care of by regulating works at the dam. With the storage basin completely filled, as it would have been in the rainy season, imagination can picture what the effect of 150 000 cu. ft. per sec. of flood waters (which has been recorded) would be when poured directly into the prism of the Canal through those regulating works.

The plans proposed for taking care of the flood waters of the smaller streams, were equally futile and impracticable. At no place was the sea-level canal proposed to be more than 200 ft. wide, and taking it as a whole, even if it could have been navigated at all, by ships of any considerable size, which is doubtful, it would have been at such slow speeds as to make it practically a makeshift. As contrasted with the present lock type canal, with the latter's 1 000-ft. approaches, its 30 miles of Gatun Lake where any speed can be maintained, and with only 8 miles of a minimum width of 300 ft., there is no room whatever for argument. The high-level lock canal can and does pass ships of any size thus far afloat, much quicker than could such a sea-level canal as that proposed, even if the latter could have passed them at all, which is extremely doubtful.

It is not argued that granting a *true* sea-level canal were built, even with a tidal lock, of a width sufficient to allow a fair rate of speed for a ship moving through it, and always providing that the flood waters of the Chagres and other streams could be safely taken care of, that such a canal would not be superior as a method of transit between the two oceans. But the time required to build such a canal, and the enormous cost of it, as truly said by President Roosevelt, rendered it prohibitive. No mistake was made in the adoption of the type of canal that we have to-day.

The vulnerability of the lock type in case of an attack by an enemy has been urged against it. This criticism is quite true, and the same is true of such a sea-level canal as was proposed. Its one tidal lock would have been equally open to damage as are the locks of the present canal, and the destruction by bombing of the 180-ft. high dam at Gamboa, letting some 200 000 000 to 300 000 000 cu. ft. of water smash into the canal, would not be a pleasant thing to contemplate, and the sea-level canal as proposed, with its narrow, crooked channel, could have been easily blocked by the sinking of a ship. No canal can be made safe from damage by an enemy in war time, excepting by keeping him away from it. It must be remembered that twenty-two years ago,

bombing planes were in the future, and even so, the lock type is no more vulnerable than a narrow sea-level canal would have been.

Once the lock type of canal had been decided upon, the way was cleared so that the plans incident to its construction could be made and put into execution. The excavation of the immense amount of material in Culebra Cut was already under way and going rapidly, for, of course, it was evident that millions of cubic yards could be removed, whatever type was to be chosen. Many wash-borings had been made in the valley of the Chagres River at Gatun, and these borings proved conclusively that a stable foundation for both dam and locks was there, contrary to the vehement assertions of the sea-level advocates. As before remarked, the Gatun Dam was the keynote, the very motif of the lock type, and its adoption and construction was the great engineering accomplishment of the enormous project of the Canal. It is, of course, true that hundreds of other important engineering problems remained to be solved, but they were more in the nature of routine, and precedents existed to guide in their solution. Gatun Dam and Locks constituted one controlling feature in the time and success of building the Canal; and the other one was the excavation of Culebra Cut, and this was primarily a transportation problem, and to a much less extent, one of engineering.

Time and space do not permit the detailing of the many—it would be uncomplimentary to call them crack-brained—schemes, which were urged upon the Chief Engineer to move these millions of yards. All such schemes were waived aside, for he knew from experience that the work could only be done in a reasonable time, and at allowable cost, by steam shovels and work trains. Material, rock or earth, can be excavated quickly, but the problem of transportation is not so easily solved. The dipper must be kept moving every possible minute, if the steam shovel was to do the work. Accordingly, the Chief Engineer devised a system of trackage, complicated from a layman's point of view, but really extremely simple, which permitted the minimum loss of time in moving the empty cars to the shovels, and in taking away the loaded ones. That was all there was to the problem. Its solution was a success, so much so that the system was continued until the entire mass of material had been removed from the Cut, and the end of work came in the very year when the Chief Engineer had predicted that it would come, and he did so in the face of criticisms and even ridicule from doubting engineering and transportation experts.

Meantime, to secure and maintain a sufficient amount of labor, skilled and unskilled, employment agencies were opened in many of the large cities of the United States. Agencies for the recruiting of common laborers were established in the British and French Caribbean Islands, and the necessary shipping arrangements were perfected. There seemed to be an endless supply of blacks to fill the requirements for common labor, but very soon it became painfully apparent that these people had gotten the totally mistaken idea into their woolly heads, that no other than negro labor could be had, and their efficiency, if such a word could be applied to them, began to decrease to an alarming degree. It became apparent that they needed an object lesson and they got it.

At first, a proposal was made to bring Cantonese coolies over from China, but this the Chinese Government, in view of the de Lesseps' experiment, would not permit. Then an attempt was made, or suggested, to get laborers from Cuba, but this raised the protests of the sugar planters and had to be abandoned. Finally, the Chief Engineer sent an agent first to Spain, from which country he was promptly thrown out, and then to Paris. From his office there, he was able, by some *sub rosa* negotiations with the steamship lines, to send over about 7 000 peasants from the Biscayan Provinces of Spain.

These men were hired at the rate of $1.60 gold per day of 8 hours, just twice what the blacks were receiving. In a few days a number of the Spaniards struck for $2.50 per day and proceeded—more or less armed—to clean the blacks out of Culebra Cut. They were met quickly by a force of the Canal Police, and a conflict was staged, during which several of the Spaniards achieved immortality, and then the mass of them went back to work, and little or no trouble was had with them thereafter. A stern lesson was needed, for if they had succeeded in terrorizing the negroes, every one of the latter would have taken to the jungle, and the labor and organization of months would have been destroyed.

All this time, houses and quarters, hotels, school-houses, churches, jails, of course, warehouses, and docks, and many other classes of structures were being built, about 75 000 000 ft. of timber, mostly brought from Puget Sound, being used in 18 months. The Commissary, operated under the Panama Railroad, was thoroughly re-organized, and constantly carried a stock of every conceivable article, of a current invoice value of nearly $1 000 000. A system of cash payments—through coupons—was put into effect. Delivery by railroad supply train to practically every family and eating place on the Zone was completed by 9:00 A. M. of each day. A refrigerating and ice plant, a laundry, and a bakery were installed in connection with the Commissary, and the employees could buy anything in the line of food from beef and pork to ice cream, and luxuries from evening clothes to face powder, and all at exact cost to the Commission, which generally meant at about 70% of the prices prevailing in the United States. Meals for the whites, such as did not keep house, cost thirty cents, and these meals equalled those furnished in this country at double the charge. It was about at this stage of proceedings, that the Chairman of the Commission sent to the Zone a party who had been operating boarding houses for laborers on some of our Western railroads. He brought a proposal approved by the Chairman, that he would take over the Commissary, and the feeding of all classes of employees, and as far as the price of meals was concerned, it was suggested that they be fixed at 33⅓% above those already in force, and, presumably, a similar approximate advance on other Commissary supplies. This brilliant business proposition was instantly vetoed and the Chief Engineer continued to administer the food supplies and the Commissary through the proper department.

Colon and Panama were provided with pure water, and with modern sewerage systems, and each city was paved to a large extent with vitrified brick brought from Illinois. Some twenty or thirty small towns were built along

the line of the Canal, and all were provided with modern sewerage and water systems, hard surfaced streets, sidewalks, etc. In the Canal prism, an 8-mile air pipe line was laid, supplied by two large compressor plants, to give air to the hundreds of rock drills used in Culebra Cut.

Much concrete was used, the aggregate for which was furnished by crusher plants located at convenient places. A location was secured from the Panamanian Government, about twenty miles along the coast from Colon, where an immense gyratory crushing plant was installed to furnish the crushed rock for the Gatun Locks and other construction at that end of the Canal. Engineers experienced in such work were placed in charge of the design of the locks and their many adjuncts, and this important feature of the project was well under way in the early months of 1907, these same engineers continuing their tasks throughout the entire time of construction of the Canal.

It would be futile to further attempt to name the many other necessary accessories which had to be designed and built to carry on the construction work of the Canal. The recital here of all these activities may seem tedious, but they are briefly touched upon because they were carried out mostly by members of the American Society of Civil Engineers. It will be recognized that engineering as carried on in the multiplicity of requirements on the Canal Zone, was a very comprehensive term. It was a truly paternal job for the Chief Engineer. One hour he would be studying and deciding on some important matter, and the next hour trying to soothe the grief of some matron who complained that so and so, the wife of another employee who received the same salary as her husband, had one more rocking chair than she had, all quarters being furnished.

The work of rehabilitating, double-tracking, and otherwise enlarging the facilities of the Panama Railroad, which, in addition to moving a large commercial business, was utilized to handle all work trains, including the hundreds in service at Culebra Cut, was in progress. The management and operation, on the Isthmus, of the Panama Railroad Steamship Company came under the authority of the Chief Engineer, as also did the handling and stevedoring of the ships of some dozen of foreign lines.

An erroneous idea long prevailed and likely does yet, with the general public and some engineers, that the work of these two years covered only that of preparation—far from such was the case. While the work of preparation was being pushed, so also was the actual construction of the canal proper. The dredging of the ocean approaches, the excavating for the locks at Gatun and Pedro Miguel, the construction of the dams at both places, was actively proceeding. Many steam shovels, drills, and work trains were successfully at work in Culebra Cut, so that during the early part of 1907, the material removed from the Cut averaged about 750 000 cu. yd. per month. The whole elaborate machine which had been built up so quickly and carefully, was working smoothly, and the output of all classes of construction was increasing daily at a rate that exceeded even the sanguine expectations of the Chief Engineer. The hardest problems were solved, the Rubicon was crossed, the Canal was being built, and everything was set for its completion in the time pre-

dicted by the Chief Engineer. Only gross mismanagement or a failure to supply the necessary funds, could militate against its triumphant accomplishment.

Much to his surprise the Chief Engineer was appointed Chairman of the Isthmian Canal Commission, taking up its duties in addition to those already in his charge. This appointment was justly regarded by him as an expression of approval, on the part of the President, of the Chief Engineer's work and a recognition that the successful construction of the Canal was assured.

Not long afterward, knowing full well that there had been carried out—even beyond the letter—the promise made to the President and knowing that the latter was alive to the fact, I resigned from the service of the Canal. Many reasons, mostly from irresponsible scribblers, were given as the cause of this resignation. In one particular the reasons alleged were alike—they were all absolutely false. Not the slightest friction ever developed between the Chief Engineer and the President, or, in fact, with any one, high or low, connected in any manner with the enterprise. The reasons for the resignation were purely personal, and involved nothing in regard to the work nor with any parties associated in it.

I have never declared these reasons, and probably never will, as they were private and of no particular interest to the public. I have a petition signed by thousands of the white employees then on the Canal Zone, asking me to withdraw my resignation, and promising to work even harder to push the construction to completion; but it could not be. The expressions of regret which greeted me from the immense number who gathered to bid me farewell, a gathering which in size probably exceeded any function ever held on the Isthmus, were sincere and fervent. I was pleased to receive them, and to know that personally as well as officially, my work had borne fruit; and I felt well assured that the work which had been so near my heart, had been given into competent hands, as the future proved in every way to be the case.

Last February, it was my privilege and pleasure to visit the Canal Zone, the first visit since my resignation in 1907. While there I was the guest of Governor M. L. Walker and his charming family, and the hospitality and reception which I met from them, and from the entire entourage on the Zone, was overpowering. Governor Walker and I were friends of many years' standing, and it was no surprise to note the perfect manner in which the operation of the Canal and of its many adjuncts was being carried on, due to his administrative ability and his able staff. Receptions, dinners, and social functions were showered on me, and best of all, I was made to realize by high and low—by all classes—civil and military—that the part I was called upon to take in the creation of the Canal was not forgotten. It was an experience the recollection of which will linger as long as life lasts.

The daylight hours of those ten days were spent in examining in detail practically every feature of the Canal. Ocean approaches, marine and railway terminals, dams, locks, Culebra Cut, nothing of consequence was overlooked; and the general result was that the whole machine was the realization of a

dream of twenty years before, only the realization was more perfect than I had dared to hope.

It would be trite and quite unnecessary here to describe in detail the operation of passing ships through the locks. It is not much less than a marvel to see what perfect control is had over every movement, and how quietly and quickly those movements are performed. The locks fill and empty—each movement—in 7½ min., and it fills one's mind with something like awe, when he sees that by the movement of a switch, no larger than one's finger, a 25 000-ton ship rises, or falls as the case may be, as easily as one would juggle a jackstraw. First-class machinery and perfect co-ordination of all parts, with intelligent trained operatives, does the work, and forcibly demonstrates how man's engineering talents can make the mysterious forces of Nature the servants of man's will.

Much time was given to examination in detail of the locks, their chambers, gates, motors, signals, control towers, emergency dams, in fact, of all their parts. The process of locking ships through was fascinating to watch, the ease with which it was done and the silence of it all was impressive. A trip on a ship from Buenos Aires en route to San Francisco carried me through the whole length of the Canal. Also in company with Governor Walker, some hours were spent in a launch examining Culebra Cut. An interesting four hours was taken at Gatun Dam and Locks. Most engineers, at least know the dimensions of that dam—8 400 ft. in length, ½ mile wide (thick) at its base, and sloping gently to a width of 100 ft. at its crest. The top of the dam is 105 ft. above sea level, or 20 ft. above the normal level of Lake Gatun, which it forms. It is an earth dam, planted with grass and shrubbery, and has the unique distinction of being the only dam in the world having a golf course on it, a standard length nine-hole course. Employees refer to the dam as "our ten million-dollar golf course".

At the spillway is located the plant which generates the power and light for the Canal and its signals; light for all shops, offices, residences, and quarters; power and light for fortifications; in fact, for each and every electrical need in the Canal Zone. In addition, there is at Miraflores a very large installation of steam and Diesel electric generating stations, so that if trouble occurs at the hydro-electric plant at Gatun, a switch throws the entire load on the emergency plant at Miraflores with no delay whatever.

But all such descriptions are merely threshing over old straw, and it is needless to devote time and space to even a recapitulation of the most important features of the Canal, they must be seen and studied with an experienced eye to be appreciated. Suffice it to say, that in one man's mind at least, and his opinion is shared by thousands who are qualified to speak, it is as well nigh as perfect a machine, in design, in construction, and in operation, as the genius of man can plan and execute.

The work of Maj.-Gen. George W. Goethals, U. S. A. (*Retired*), M. Am. Soc. C. E., former Chief Engineer, and of his associates on the Zone, cannot be given too high credit. The details of the Canal and of its accessories have been wrought out and completed marvelously well. Nothing necessary to make a complete and perfect whole has been forgotten, and the character of the

work, as regards material and execution, is superb. No mistake was made in entrusting the work to their hands, and while others might have done it as well, I feel that they could not have done it better, and I take this occasion to pay a high and well deserved compliment to Gen. Goethals and his associates.

The average time of passage of ships through the Canal is 8 hours. Its generous widths would permit of shortening this time if occasion required. It can handle easily 48 ships per day, or more than 17 000 per year. It is 42 ft. deep in the Atlantic and Pacific sea-level section, from 45 to 85 ft. deep in Gatun Lake where it is 500 to 1 000 ft. wide, and 40 ft. deep and 300 ft. wide in Culebra Cut. It has large coal and oil storage plants, extensive marine shops, and one of the largest dry docks in the world. There are modern hotels, restaurants, laundries, ice and refrigerating plants, churches, schools, clubs, places of amusement, and Balboa, where the Administration Headquarters are located, is one of the loveliest places for residence that I have ever seen.

The Canal tolls are levied on the net tonnage of the ships and range from 25.7 cents upward, according to capacity and not the nature of the cargo, but of the class of ship. The average toll for bulk cargoes is about 65 cents per long ton. The cost to ships using the Canal is approximately equal to six days at sea; of course, this varies with the class of ship and her speed.

As a concrete example: A steamship passing through at the time of my visit, carrying passengers and a limited amount of freight, New York to San Francisco, tonnage, 22 500, speed about 14 knots, costing for overhead, depreciation, maintenance, and operation not far from $2 000 per day, saved about 8 000 miles, or some 23 days, by using the Canal instead of the Straits of Magellan. The tolls were $11 000, saving $35 000, besides which, that line would not be sending ships around the world carrying passengers if the Magellan route was obligatory.

Periodically, the capacity and the water supply for lockages of the Canal have been questioned, recently with an insistence and a demand which has already reached the President, and probably will reach Congress, that a canal at Nicaragua be built without delay to relieve the situation. Let us look into what, taking these demands at their face value, would indeed be a very vital matter. Thus far, the water supply has proven ample. Gatun Lake covers an area of 163 sq. miles and it is entirely practicable to raise its normal level 6 in. and possibly 1 ft., to elevations of 85.5 or 86 ft., which would add 52 160, or 104 320 acre-ft., respectively, to the present supply of water. The water-shed of this lake covers an area of 1 350 sq. miles, and from this immense water-shed there flows into the lake a number of large streams, on many of which storage reservoirs to conserve the flood waters which descend frequently, can be built.

But plans for a very large increase in the present supply were made long since, in fact, some years before the type of canal was selected. About 6 miles up the Chagres River, at Alhajuela, there is a natural site for a storage dam which would impound flood waters to an amount nearly equal to the present supply. This dam can be built in 3 years, or less, and would cost approxi-

mately $8 000 000 at present prices of labor and material. With the additional supply thus afforded, all fears of a shortage of water could be dismissed for many years to come. It is true that given a season when the rainfall is abnormally light, the necessity for conserving the supply is apparent. A large quantity of water is now used to create power to generate electricity, but in case of necessity this water could be saved by running the steam-generating plants previously described. Twice I visited the site of the proposed Alhajuela Dam in 1905, and made a thorough examination of its availability, and I know that it is wonderfully well adapted for the purpose planned.

As a measure of insurance, the construction of this storage dam should be begun as soon as it can be authorized, although the increase in traffic must be greater than it has been for some years, before any serious apprehensions need be felt on account of a shortage of water.

The Canal is operated only during daylight hours, but it can be readily operated during the night, as it is perfectly lighted throughout its entire length. It has been noted that the nominal daily capacity of the Canal is 48 passages, but on January 17, 1924, a total of 57 vessels made the transit. In 1923, a total of 5 466 ships passed through, and, in 1926, a total of 5 964, an increase in 3 years of 498, or an increase of 9% in 3 years, or an average increase each year of 3%, which does not seem to bear out the claims made by irresponsible parties as to tremendous increases in traffic.

The daily average number of ships passing through the Canal during May, 1926, was 15.9 and for 11 months, ending with April, the highest number passing in any one day was 16.3, the average number for that period being 14.97. Therefore the statement that the Canal is not and has not been worked to more than one-third of its capacity, is ultra-conservative, especially so when it is considered that 57 ships have passed through in 1 day without difficulty.

The capacity of this Canal should not be measured by dead tonnage, the number of ships passing through is the real yardstick. Copper-laden ships from Chile, lumber-laden ships from Puget Sound, and oil tankers from the Pacific Coast, all load heavily and, in locking, a ship is a ship in point of time, regardless of its tonnage.

The Sault Ste. Marie ("Soo") Canal at the outlet of Lake Superior, during the 234 days when it was open in 1926, passed 21 000 vessels of all sizes, or an average of 90 per day, and its tonnage for 1926 was about 86 000 000. About 92% of this large traffic is handled by the American locks, and the greater part of this through the third and fourth (two in number) locks. If the "Soo" Locks could work the entire year, as can those at Panama, they would, on the basis of 1926, pass 32 850 vessels, or about 134 000 000 tons. The Panama Canal passed 26 836 241 tons in 1926, exceeding that of the sea-level Suez Canal. In view of the ease with which the "Soo" Canal could pass more than 100 000 000 tons yearly, if not icebound for four and one-half months, what could the Panama Canal pass in twelve uninterrupted months, with practically the same facilities as the "Soo". And yet, reliable authority declares that the "Soo" Locks are not worked to more than 50% of their ultimate capacity.

The four-year period from 1923 to 1926 shows over the preceding period an increase of 87 per cent. The maximum number of ships passing through the Canal in any one month was 611, in March, 1927, or an average of 19.7 per day, or an average of not over 40% of its capacity.

Liberty is here taken of quoting Governor Walker, who is probably the best informed man in the world as to the actual situation:

"I do not think that we should take the increases shown above too seriously as a basis for predicting future traffic through the Canal. It must be kept in mind that, due to the World War, routing of all shipping was disturbed, and it was a matter of years before normal trade routes could be re-established. The phenomenal growth of traffic through the Canal between 1919 and 1923 can be largely attributed to this re-adjustment. It should be noted that the increase between 1923 and 1926, four years, is only 7.6% for commercial carriers, and it should further be noted that 25% of our business now is incident to the transfer of oil from the Pacific Coast of the United States to the Atlantic Coast, and that this may cease more or less entirely due to the production of oil on the north coast of South America, and a revival of activity in the Mexican fields."

But thus far, the whole story of capacity has not been told. The present double, or twin, locks of the Panama Canal were located and built so that when the need arose for greater capacity than they afforded, a third set of locks could readily be added alongside the present ones. Probably in some years to come it may be advisable to add these locks, which can be done without disturbing those now in service. The present locks are 1 100 ft. long, 110 ft. wide, with a draft of 41 ft. over the miter-sills. These locks safely and quickly have transited the largest, that is, the widest ship afloat, and can do the same with even wider ones. But probably it would be wise, as a measure of prudence, to design the new locks of larger capacity, and of deeper draft, although it must be remembered that when ships are built requiring more than 40 ft. of water, many of the most important harbors of the world must be deepened accordingly.

Again quoting Governor Walker:

"Since we are running now, excepting when we put the United States fleet through, at about 40% of our capacity, the third lock with additional water storage will very nearly double this capacity. I cannot see any future for the Nicaraguan Canal."

Again:

"In my opinion the correct policy now is to proceed at once with the construction of the Alhajuela storage project, to watch the situation and commence the installation of the third lock when indications warrant and to entirely dismiss the Nicaraguan Canal from consideration for a long time to come."

In which opinion and conclusions I, who have watched and studied the situation closely for the past four years, most emphatically agree. The more than $500 000 000 which a canal at Nicaragua would cost (and, by the way, it would be a lock canal requiring a lift 30 ft. higher than the present Panama Canal because Lake Nicaragua, the key to the situation, is 115 ft. above sea level), would practically be wasted, and if spent at all, had better be spent

in our country where vast problems confront us, and with which only National resources can cope. The addition of the third lock and the increased water supply at Panama which the storage dam at Alhajuela would afford, as far as human eye can look into the future, would give the Canal ample capacity for all transits for the next fifty years, and perhaps longer.

To any one who thinks that the capacity of a lock canal is very limited, a study of the "Soo" Canal would be profitable. This canal carries the great bulk of the tonnage that passes Detroit, and yet three years ago (later data are not available) more tonnage passed Detroit in eight months of open navigation than the total of all tonnage that passed through both the Suez and the Panama Canals in two years.

As previously remarked in discussing the relative merits of a sea level *versus* a lock canal, I am not in the least arguing that a lock canal is superior to a real, *bona fide* sea-level canal, such a one as our erratic engineering friend, Bunau-Varilla—one of de Lesseps' Chief Engineers—termed the "Straits of Panama". And speaking of Bunau-Varilla, one is reminded of the definition of a philanthropist, one "who is always ready to spend the other fellow's money."

There has always been a mistaken idea with the general public, as well as with some engineers, about the so-called sea-level canal as proposed by the majority of the Consulting Board. If built, it would have been a useless, expensive toy to be flung sooner or later on to the National scrap pile. I have been privileged to be of some little service to my country, and the greatest service I ever gave to it, was the part I took in preventing foreign votes from foisting a nameless, useless thing upon a too credulous American people.

Probably the most of the critics of the present Canal are sincere in their criticisms, but many of their criticisms show a decided lack of knowledge of conditions. As far as the danger to it from earthquakes is concerned, Mother Earth is a nervous old lady, and when and where and how hard she will wiggle the next time is beyond the ken of the wisest, as she is probably wiggling somewhere all the time. The United States is not free from her vagaries. The Pacific Coast at San Francisco, the Atlantic Coast at Charleston and in New Jersey, the mountain ranges of Montana, and even staid old New England, have felt the rattling of her old bones. Panama may have quakes, and like most parts of the earth she probably will have, but it is reasonably certain that no severe ones have occurred there during the past 200 years, or some fragile masonry structures known to have been built even longer than that, would not now be standing practically as they did when their long-forgotten builders completed them.

Ninety-nine times out of one hundred when the words, "Panama Canal", are spoken, the question is asked, "How about the slides?" This same subject has been the windmill against which zealous Don Quixotes have been tilting for the past fourteen years. That there have been serious slides is well known, and it is within the realms of possibility that there may be others. As noted previously, I took occasion during my recent visit to the Canal, to examine Culebra Cut, and my opinion is, that while slides may occur, there was not to my eye any reason to believe that they would be serious enough to block the

passage of ships, for it would take a very large slide to close the 300-ft. wide channel which exists throughout the whole length of the Cut. Enormous dredges with ample other floating equipment are at hand, and the channel is kept clear. Criticism is heard that the slopes of the Cut should have been made flatter—this is a matter of opinion. It is a well-known axiom among railroad construction engineers that, in a cutting where the material is earth, or earth and rock, especially where the rainfall is heavy, the steeper the slope can be safely made, the less cost of maintenance.

The flatter the slopes in such cases, the more area is exposed to rainfall, and the more loose material is washed into the cutting. And at Panama, where the rainfall is always heavy, where torrential rain has been known to fall at the rate of 1½ in. per hour for 10 consecutive hours, this was a factor to be considered. I do not know what determined Gen. Goethals to use the slopes that he did, but if he decided upon them as the result of his best judgment, he did what was exactly right and just what I would have done if I had been responsible for results.

There is another angle to the situation. Granting that approximately as much material has been taken out of the Cut since the opening of the Canal, as was taken out of the original prism during construction, it is probable that the removal of this additional yardage has cost much less, possibly only one-half as much as it would have cost if taken out before the opening of the Canal. Dredging is one of the very cheapest methods of handling such material.

There are some who criticize the sun, which is now almost at the peak of one of the periodic phases which come upon it every eleven years. One of the symptoms is an increase in the number of spots; another is a slight increase in the average heat of its rays; another important symptom is that the percentage of the ultra-violet rays is far above normal. But the sun keeps right on shining and fructifying the earth, giving us everything that we have, even life itself, and will probably continue to do so for some time to come.

The three Chief Engineers who were responsible for the planning and construction of the Panama Canal, were Members of the American Society of Civil Engineers, as were also many of their subordinates. The Canal is very largely the creation of this Society, and as such its members should take a just pride in it, as being a marvelous triumph of man over the hostile forces of Nature, and a work which, from an engineering and business point of view, can truly be classed in a rank by itself, a rank which has never been reached before since the dawn of civilization.

And in closing this sketchy review of a few of the activities in which I had a part, and particularly in consequence of the demands which are being made for something different, unnecessary, and something which the most of its proponents do not themselves understand, we may call to mind and apply the words of wise old Omar, the Persian poet-philosopher:

> "Some for the Glories of this World; and some
> Sigh for the Prophet's Paradise to come;
> Ah, take the Cash and let the Credit go,
> Nor heed the rolling of a distant Drum."

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1651

THE STRESSES IN A FREE PRISMATIC ROD UNDER A SINGLE FORCE NORMAL TO ITS AXIS*

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With Discussion by Messrs. William R. Osgood, Henry D. Dewell, A. C. Schwager, and Joseph N. Le Conte.

Synopsis

The resistance of engineering structures to earthquake forces has been studied by engineers in the past but revived interest in the subject is evidenced since the Japanese earthquake of September, 1923, and the Santa Barbara earthquake of June, 1925. The Japanese seismologists and engineers, in particular, have been making intensive studies of the effect of earthquake shocks on structures for many years, and the results of their studies have been published in the *Transactions* of the Seismological Society of Japan. The writings of the late F. Omori, in particular, have become classics in earthquake literature.

Omori observed that in general tall chimneys did not overturn, nor fracture at the base, but were ruptured at a point about two-thirds of their height. To this point of rupture he applied the term, "center of percussion," which term is used in the science of physics, and is associated with the action of an impulsive force acting upon a rigid body free to move. Hence, the few writers on the subject of the resistance of engineering structures to earthquakes have assumed, in general, that, under the influence of earthquake shocks, tall, relatively slender structures act differently from low structures of relatively broad lateral dimension. Some of these writers, under the influence of Omori's statements, have assumed that the computed lateral force of the earthquake shock must be doubled for impact, in the case of high slender structures. There is evidence, also, that the distribution of

* Published in January, 1927, *Proceedings*.

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stresses in such structures is not generally understood. Indeed, there is nothing in Omori's published writings to indicate that he found the distribution of bending moments and shears in a structure which exhibited the phenomenon of rotation about an instantaneous center.

In designing structures to resist earthquake forces, American engineers in general have been content to use, in lieu of an assumed earthquake horizontal force, a wind pressure which, in their opinion, was equivalent in effect to the shock of the earthquake. Some engineers, in designing high reinforced concrete chimneys, have computed the total force acting on the chimney as a whole from the assumed horizontal component of the acceleration and have then increased, or decreased, this basic acceleration along the height of the chimney to cover the unknown bending moments and shears which would result from a force causing the phenomenon of "center of percussion."

This paper sets forth, as a problem of pure mechanics, the distribution of bending moments and shears in a slender rigid prism, free to move under the action of a single impulsive force. It is found that such prism under the action of such impulsive force does tend to turn about a center of instantaneous rotation, bearing out the observations of Omori, and that a point of maximum shear occurs at such center. Formulas are given for determining the location of such center of instantaneous rotation and the distribution of shears and bending moments.

The discussion is illustrated by practical applications to specific examples, and graphs of the resulting shears and bending moments are shown.

The following analysis was undertaken by the writer as a problem in pure mechanics, without thought of its application to practical construction, but since the Santa Barbara earthquake of June, 1925, attention has been called to the fact that the formulas developed might have some bearing on the design of tall slender structures. The problem herein investigated is concerned with the stresses in a rigid prismatic rod acted upon by a single impulsive force normal to its axis when the rod has no support whatever except the yielding support of inertia. It is important to note that the analysis cannot apply as an exact quantitative method to the stresses in an engineering structure, as the elasticity of such a structure greatly reduces the magnitude of the stress. The formulas, therefore, are of qualitative rather than of quantitative value.

Let $A\,E$, Fig. 1, be a rigid prismatic rod free of all constraint. If m is its mass and l its length, its moment of inertia about an axis through the center of mass and normal to its geometric axis will be:

$$I_G = m\left(\frac{l^2}{12} + k_0{}^2\right) \qquad (1)$$

in which, k_0 is the radius of gyration of its areal cross-section about a parallel axis through its centroid. Now, if for such a rod, K, be the mass per unit length, $m = K\,l$, and Equation (1) becomes:

$$I_G = K\,l\left(\frac{l^2}{12} + k_0{}^2\right) \qquad (2)$$

Imagine a force, P, applied at any point, D (Fig. 1), a known distance, x, from G, the center of mass. Imagine this force to act for a very short time, a time so short that at its conclusion the angular velocity of the rod is still very small, and the normal acceleration of the mass, that is, the acceleration of particles in the direction, EA, is negligible. It is not necessary, however, that the force, P, be constant during this brief time, but it may follow any time law whatever. Such a force may be considered to act during any phase of an impulsive blow. The problem now is to determine the distribution and magnitude of the bending moments and shearing stresses along the rod as functions of P and physical constants.

First the instantaneous motion of the rod as a whole in terms of P and of the time, t, must be determined. Consider the rod hinged so as to rotate about a fixed rigid axis normal to the paper at O (Fig. 2). Then the rod is constrained, and while the force, P, is acting there will be in general a bearing reaction, B, at O parallel with P, since P has no component along the rod, since gravitation is neglected, and since the acceleration of G along the axis of the rod is so small in the short time considered as to be negligible. Taking forces to the right as positive, and counterclockwise rotation positive, from the principle of impulse and linear momentum:

$$-\int_0^t P\,dt + \int_0^t B\,dt = mv \qquad (3)$$

v being the velocity of G at time, t. Also, from the principle of moment of momentum,

$$-\int_0^t P\,(x + r)\,dt = I_0\,\omega \qquad (4)$$

O being at rest. The kinematic relation between v and ω is,

$$-v = -r\omega \qquad (5)$$

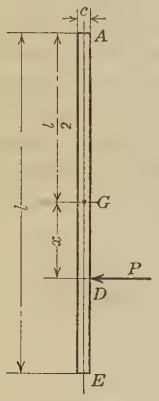

Fig. 1.

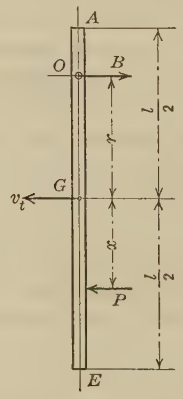

Fig. 2.

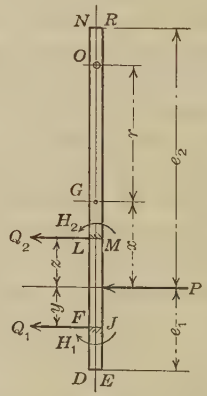

Fig. 3.

Remembering that $x + r$ is a constant as regards time, Equation (4) gives,

$$-\int_0^t P\,dt = \frac{I_0\,\omega}{x + r} \qquad (6$$

Substituting from Equation (6) and from Equation (5) in Equation (3),

$$\frac{I_0\,\omega}{x+r} + \int_0^t B\,dt = m\,r\,\omega$$

and differentiating,

$$\frac{I_0}{x+r}\,d\omega + B\,dt = m\,r\,d\omega \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Equation (6), however, when differentiated, gives,

$$-P\,dt = \frac{I_0}{x+r}\,d\omega \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Substituting for $d\omega$ in Equation (7), its value from Equation (8), thus eliminating dt,

$$B = P\left(1 - \frac{m\,r\,(x+r)}{I_0}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Equation (9) shows that within the short time limit considered, B is directly proportional to P, regardless of the law of variation of P. B of course will be zero if P is zero, but even if P has any finite value, B will be zero if $m\,r\,(x+r) = I_0$. Now,

$$I_0 = I_G + mr^2$$

hence, if B is to be zero,

$$m\,r\,x + m\,r^2 = I_G + m\,r^2$$

$$r = \frac{I_G}{m\,x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

If O is located at the distance, r, given by Equation (10) there will be no bearing reaction at O. The axis can then be removed, and O becomes the free axis of rotation.

Substituting in Equation (10) the value of I_G given in Equation (2):

$$r = \frac{1}{12\,x}\left(l^2 + 12\,k_0^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

This important relation shows that if any force, P, be applied normal to the axis of a rigid prismatic rod of length, l, at a point distant x ft. from G, the position of the center of free rotation can be expressed in terms of l, k_0, and x only by Equation (11).

It is interesting to note that if the force is applied at the end of the rod, $x = \frac{l}{2}$ and $r = \frac{l}{6} + \frac{2\,k_0^2}{l}$. Furthermore, if the rod is slender so that k_0^2 is negligible compared with l, $r = \frac{l}{6}$, or the axis of free rotation is two-thirds the length of the rod from the end where the force is applied. Also, if $x = 0$, $r = \infty$, as is obvious. Finally, if P is known as a function of t, the motion of the rod is completely determined by Equation (8). If in Equation

(8) for I_0 its equivalent, $I_G + mr^2$, is substituted, and for r its value for a center of free rotation given in Equation (10), or $r = \frac{I_G}{m\,x}$, Equation (8) becomes,

$$d\,\omega = -\frac{P\,x}{I_G}\,dt$$

$$= -\frac{12\,P\,x}{K\,l\,(l^2 + 12\,k_0^2)}\,dt \qquad (12)$$

Proceed now to determine the internal stresses. If a part of the rod be imagined as cut away by any plane normal to the axis, then the force and couple which must be applied to that section to cause it to move as indicated by Equation (12) will represent the shearing stress and bending moment in that section. Consider the stresses acting on the piece below the plane, $F\,J$. These will consist of a force, Q_1, and a couple, H_1, in the directions shown, Fig. 3. The linear momentum of $F\,J\,D\,E$ gives:

$$-\int_0^t Q_1\,dt = m_1\,v_1 = K\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right)\omega$$

The moment of momentum about the center of mass of the same piece gives:

$$-\int_0^t H_1\,dt + \int_0^t Q_1\,\frac{e_1 - y}{2}\,dt = I_G'\,\omega = K\,(e_1 - y)\left(\frac{(e_1 - y)^2}{12} + k_0^2\right)\omega$$

Differentiating the first of these latter equations, and substituting for $d\,\omega$ its value from Equation (12), there results by the elimination of $d\,t$,

$$Q_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right) \qquad (13)$$

This is the shearing stress in terms of y, and is a parabola.

Differentiating the second, making the same substitutions, and also substituting for Q_1 its value from Equation (13),

$$H_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right)\frac{e_1 - y}{2}$$

$$+ \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(\frac{(e_1 - y)^2}{12} + k_0^2\right)$$

or,

$$H_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left[\left(r + x + \frac{e_1 + y}{2}\right)\frac{e_1 - y}{2}\right.$$

$$\left. + \frac{(e_1 - y)^2}{12} + k_0^2\right] \qquad (14)$$

This is the bending moment in terms of y and is a cubic curve.

Again, consider the stresses in the plane, $L\,M$, acting on the section, $L\,M\,N\,R$. Call the force Q_2 and the couple H_2, in directions shown. Let z be the distance of $L\,M$ from P, and e_2 the distance of P from the top of the bar. The equations are:

$$-\int_0^t Q_2\,dt = m_2\,v_2 = K\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right)\omega$$

and,

$$\int_0^t H_2\,dt - \int_0^t Q_2 \frac{e_2 - z}{2}\,dt = I_G''\,\omega = K\,(e_2 - z)\left(\frac{(e_2 - z)^2}{12} + k_0^2\right)\omega$$

Differentiating now and making the same substitutions as before, there results,

$$Q_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right) \ldots\ldots\ldots (15)$$

the shearing stress in terms of z, a parabola; and,

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left[\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} - k_0^2\right] \ldots\ldots\ldots (16)$$

the bending moment, a cubic curve.

Putting $y = 0$ in Equation (13) gives the shearing stress at the point of application of P necessary to give the required linear acceleration to the center of mass of the lower part; thus,

$$Q_1' = \frac{6\,P\,x\,e_1}{l\,(l^2 + 12\,k_0^2)}\,(2\,r + 2\,x + e_1) \ldots\ldots\ldots (17)$$

Again, putting $z = 0$ in Equation (15) gives the stress at P necessary to give linear acceleration to the center of mass of the upper part; thus,

$$Q_2' = \frac{6\,P\,x\,e_2}{l\,(l^2 + 12\,k_0^2)}\,(2\,r + 2\,x - e_2) \ldots\ldots\ldots (18)$$

$Q_1' + Q_2'$ should equal P, which is seen to be true, when Equations (17) and (18) are added, note being taken of the fact that $e_1 + e_2 = l$; $e_1 - e_2 = -2\,x$; and $r = \dfrac{l^2}{12\,x} + \dfrac{k_0^2}{x}$.

Also, putting $y = 0$ in Equation (14) gives the couple in section at P which, together with Q_1', is necessary to give the entire lower part the required angular acceleration; thus,

$$H_1' = \frac{12\,P\,x\,e_1}{l\,(l^2 + 12\,k_0^2)}\left[\left(r + x + \frac{e_1}{2}\right)\frac{e_1}{2} + \frac{e_1^2}{12} + k_0^2\right] \ldots\ldots (19)$$

Finally, put $z = 0$ in Equation (16) and get the couple in the plane of P, which, together with Q_2', is necessary to give the entire upper part the required angular acceleration; thus,

$$H_2' = \frac{12\,P\,x\,e_2}{l\,(l^2 + 12\,k_0^2)}\left[\left(r + x - \frac{e_2}{2}\right)\frac{e_2}{2} - \frac{e_2^2}{12} - k_0^2\right] \ldots\ldots (20)$$

$H_1' - H_2'$ should be zero, and this is found to be true by substituting for e_1 its equal, $\dfrac{l - 2\,x}{2}$, and for e_2 its equal, $\dfrac{l + 2\,x}{2}$, and subtracting Equation (20) from Equation (19).

Now is there a point in the rod where either the shear or bending moment reduces to zero? Take, first, Q_1. Let,

$$Q_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right) = 0$$

If $x = 0$, $r = \infty$, and the expression is indeterminate. However, if for r its equivalent, $\frac{1}{12\,x}\,(l^2 + 12\,k_0^2)$, is substituted and the expression reduced,

$$Q_1 = \frac{P}{l\,(l^2 + 12\,k_0^2)}\,[(e_1 - y)\,(l^2 + 12\,k_0^2 + 12\,x^2 + 6\,x\,[e_1 - y])]$$

and now putting $x = 0$,

$$Q_1 = \frac{P\,(e_1 - y)}{l}$$

which, in general, is not zero unless $y = e_1$. In other words, if the force, P, is applied opposite the center of mass, the shear is given by $Q_1 = \frac{P\,(e_1 - y)}{l}$, and this is not zero except at the ends. The shear at the centroid itself is $\frac{P}{2}$, since when $x = 0$, $e_1 = \frac{l}{2}$. If x is not equal to zero, the shear is zero at the lower end in any case, since there $y = e_1$. Putting $r + x + \frac{e_1 + y}{2} = 0$, and solving for y gives $y = -(2\,r + 2\,x + e_1)$, which being negative does not fall within the limits of the piece under consideration. Hence in the lower part the shear cannot be zero except at the lower end.

Now, put,

$$Q_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right) = 0$$

Here, if $x = 0$, the result is the same as in the preceding case. If x is not zero, Q_2 becomes zero when $z = e_2$, or at the upper end in any case. If, however,

$$r + x - \frac{e_2 + z}{2} = 0$$

$$z = 2\,r + 2\,x - e_2 \qquad (21)$$

and this may have a definite positive value, showing that in the general case there is a point of no shear in the upper part if $e_2 < 2r + 2x$.

Consider the bending moments. Put,

$$H_1 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_1 - y)\left[\left(r + x + \frac{e_1 + y}{2}\right)\frac{e_1 - y}{2} + \frac{(e_1 - y)^2}{12} + k_0^2\right] = 0$$

Substitute for r its value, namely, $\frac{1}{12\,x}\left(l^2 + 12\,k_0^2\right)$, then put $x = 0$, giving the expression for the bending moments, H_1, when P is applied opposite the center of mass,

$$H_1 = \frac{P\,(e_1 - y)^2}{2\,l}$$

This cannot be zero unless $y = e_1$. Hence, if the force is applied opposite the center of mass the bending moments will be finite throughout except at the ends. At the center of mass itself the bending moment will be,

$$H_1 = \frac{P\,e_1^2}{2\,l} = \frac{P\,l}{8}$$

Consider the expression for $H_1 = 0$ in its general form. This will be satisfied if $y = e_1$, or at the lower end. Now, put,

$$\left(r + x + \frac{e_1 + y}{2}\right)\frac{e_1 - y}{2} + \frac{(e_1 - y)^2}{12} + k_0^2 = 0$$

For any positive value of y less than e_1 the left-hand side of the equation has a positive finite value, and hence cannot be zero. It is evident, therefore, that the bending moments cannot be zero in the lower part except at the lower end.

Consider the bending moments in the upper part,

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}(e_2 - z)\left[\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} - k_0^2\right] = 0$$

If the force, P, is applied opposite the center of mass, or $x = 0$, the expression reduces to the same form as did H_1. For the general case if $z = e_2$, H_2 is zero, or the bending moment is always zero at the upper end. H_2 is also zero if,

$$\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} - k_0^2 = 0$$

This is a quadratic equation, the final solution of which is,

$$z = \frac{1}{2}\left[3\,(r + x) - e_2\right] \pm \sqrt{\frac{9}{4}(e_2 - r - x)^2 + 6\,k_0^2} \quad \ldots\ldots\ldots (22)$$

There are two real roots for Equation (22), but only one of them, namely, that obtained by using the negative sign before the radical, may be less than e_2 and fall within the limit of the piece under consideration.

Consider now where the maximum shear and bending moments will come. By this is meant the true mathematical maxima, and not the values at the point of application of P which may be numerically larger than the mathematical maxima. Evidently, there is no such point in the lower part, since the two functions increase continuously from zero at the lower end to their greatest values at the point of application of P. In the upper part, however, from Equation (15):

$$Q_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right)$$

Let $e_2 - z = \alpha$; then $z = e_2 - \alpha$ and $\frac{e_2 + z}{2} = e_2 - \frac{\alpha}{2}$. Call $e_2 - r - x = A$, a constant for the differentiation; then,

$$Q_2 = -\frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\left(A\,\alpha - \frac{\alpha^2}{2}\right)$$

Then,

$$\frac{d\,Q_2}{d\,\alpha} = -\frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(A - \alpha) = 0$$

hence,

$$A - \alpha = 0$$

or Q_2 is a maximum when $\alpha = A$, or when,

$$z = r + x \qquad (23)$$

opposite the center of free rotation. The magnitude of this maximum shear will be found by putting this value in Equation (15),

$$Q_2\Big]_{\text{max.}} = -\frac{6\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - r - x)^2 \qquad (24)$$

Again, in the upper part, from Equation (16),

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,(e_2 - z)\left[\left(r + x - \frac{e_2 + z}{2}\right)\frac{e_2 - z}{2} - \frac{(e_2 - z)^2}{12} - k_0^2\right]$$

Making the same substitutions,

$$H_2 = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\,\alpha\left[\left(-A + \frac{\alpha}{2}\right)\frac{\alpha}{2} - \frac{\alpha^2}{12} - k_0^2\right]$$

$$= \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\left[\frac{\alpha^3}{6} - A\,\frac{\alpha^2}{2} - k_0^2\,\alpha\right]$$

Then,

$$\frac{d\,H_2}{d\,\alpha} = \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)}\left[\frac{\alpha^2}{2} - A\,\alpha - k_0^2\right] = 0$$

$$\alpha = A \pm \sqrt{A^2 + 2\,k_0^2}$$

$$z = r + x \pm \sqrt{(e_2 - r - x)^2 + 2\,k_0^2} \qquad (25)$$

This gives two real values of z, but only that one obtained by using the negative sign before the radical is positive and less than e_2.

Consider a rectangular concrete column, 7 ft. long and 2 by 2 ft. in cross-section, acted upon by a force of $P = 1\,000$ lb. at right angles to the center line of one face 1 ft. from the end (Fig. 4). Then,

$$l = 7;\ P = 1\,000;\ x = 2.5;\ e_1 = 1;\ e_2 = 6;\ k_0^2 = \frac{b^2}{12}$$

in which, b is the side of the square cross-sectional area, and, hence, $k_0^2 = \frac{1}{3}$.

$$r = \frac{l^2 + 12\,k_0^2}{12\,x} = \frac{49 + 4}{30} = 1.766 \text{ ft.};\ \frac{12\,P\,x}{l\,(l^2 + 12\,k_0^2)} = 80.9$$

$$r + x = 4.266 \text{ ft.}$$

$$Q_1 = 80.9\,(1 - y)\left(4.766 + \frac{y}{2}\right)$$

$$H_1 = 80.9\,(1 - y)\left[\left(2.383 + \frac{y}{4}\right)(1 - y) + \frac{(1 - y)^2}{12} + \frac{1}{3}\right]$$

$$Q_2 = 80.9\ (6 - z)\left(1.266 - \frac{z}{2}\right)$$

$$H_2 = 80.9\ (6 - z)\left[\left(0.633 - \frac{z}{4}\right)(6 - z) - \frac{(6 - z)^2}{12} - \frac{1}{3}\right]$$

These curves are plotted on Fig. 4.

It will be noted that zero shear in the upper part comes at,

$$z = 2r + 2x - e_2$$
$$= 8.533 - 6 = 2.533 \text{ ft.}$$

Maximum shear comes at $z = r + x$, or at $z = 4.2666$ ft., opposite O. The shear at this point will be $Q = 121.4$ lb.

Zero bending moment in the upper part comes at,

$$z = \frac{1}{2}\left[3\ (r + x) - e_2\right] - \sqrt{\frac{9}{4}\ (e_2 - r - x)^2 + 6\ k_0}$$

$$= \frac{3}{2} \times 4.266 - 3 - \sqrt{6.76 + 2} = 0.440 \text{ ft.}$$

Maximum bending moment comes at,

$$z = r + x - \sqrt{(e_2 - r - x)^2 + 2\ k_0^2}$$

$$= 4.266 - \sqrt{3 + \frac{2}{3}} = 2.352 \text{ ft.}$$

The bending moment at this point is $H = -376.5$.

The formulas and relations developed thus far can be simplified to a considerable extent if the prism under consideration is so slender as to make $12\ k_0^2$ negligible in comparison with l^2. In other words, the moment of inertia of a slender rod about an axis normal to its own axis and through the center of mass is $I_G = m\ \frac{l^2}{12}$. It must be observed, however, that in developing the expressions for bending moments, the moment of inertia of a part of the prism such as $F\ J\ D\ E$ (Fig. 3) enters into the equation for the bending moment in the section, $F\ J$. Calling this I_G' and using the simplified expression,

$$I_G' = m_1 \frac{(e_1 - y)^2}{12}$$

This will not hold with the same degree of accuracy as y approaches e_1. The simplified expressions for bending moments, therefore, are not exact toward the ends. Since the stresses there are small this fault may be overlooked, and the results obtained are practically exact.

The simplified formulas are ($k_0^2 = 0$):

$$I_G = \frac{K\ l^3}{12};\quad r = \frac{l^2}{12\ x};\quad d\ \omega = -\frac{12\ P\ x}{K\ l^3}\ dt$$

$$Q_1 = \frac{12\ P\ x}{l^3}\ (e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right) \qquad (13a)$$

$$H_1 = \frac{6\,P\,x}{l^3}(e_1 - y)^2\left(r + x + \frac{2\,e_1 + y}{3}\right) \ldots\ldots\ldots\ldots (14a)$$

$$Q_2 = \frac{12\,P\,x}{l^3}(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right) \ldots\ldots\ldots\ldots (15a)$$

$$H_2 = \frac{6\,P\,x}{l^3}(e_2 - z)^2\left(r + x - \frac{2\,e_2 + z}{3}\right) \ldots\ldots\ldots\ldots (16a)$$

Now putting y and z equal to zero,

$$Q_1' = \frac{6\,P\,x\,e_1}{l^3}(2\,r + 2\,x + e_1) \ldots\ldots\ldots\ldots (17a)$$

$$Q_2' = \frac{6\,P\,x\,e_2}{l^3}(2\,r + 2\,x - e_2) \ldots\ldots\ldots\ldots (18a)$$

$$H_1' = \frac{2\,P\,x\,e_1^2}{l^3}(3\,r + 3\,x + 2\,e_1) \ldots\ldots\ldots\ldots (19a)$$

$$H_2' = \frac{2\,P\,x\,e_2^2}{l^3}(3\,r + 3\,x - 2\,e_2) \ldots\ldots\ldots\ldots (20a)$$

The zero shear in the upper part comes at,

$$z = 2\,r + 2\,x - e_2 \ldots\ldots\ldots\ldots (21a)$$

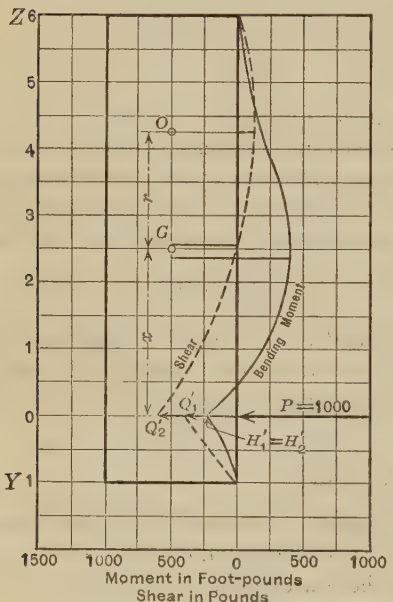

FIG. 4.

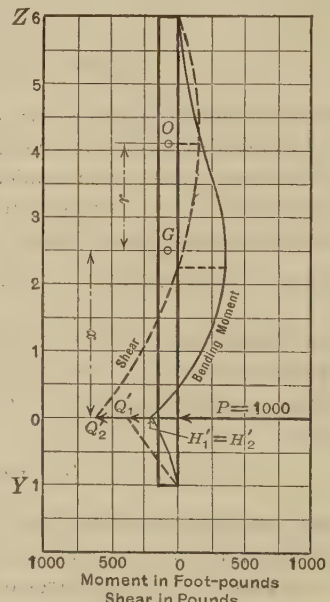

FIG. 5.

The zero bending moment in the upper part comes at,

$$z = 3\,r + 3\,x - 2\,e_2 \ldots\ldots\ldots\ldots (22a)$$

The maximum shear in the upper part comes at,

$$z = r + x \ldots\ldots\ldots\ldots (23a)$$

or opposite the center of free rotation as in the general case. The value of this maximum shear will be,

$$Q_2\Big]_{\text{max.}} = -\frac{6\,P\,x}{l^3}\,(e_2 - r - x)^2 \ldots\ldots\ldots\ldots\ldots\ldots (24a)$$

The maximum bending moment in the upper part comes at,

$$z = 2\,r + 2\,x - e_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25a)$$

opposite the point of zero shear. The value of this maximum bending moment will be,

$$H_2\Big]_{\text{max.}} = -\frac{8\,P\,x}{l^3}\,(e_2 - r - x)^3 \ldots\ldots\ldots\ldots\ldots\ldots (26a)$$

In order to illustrate the use of these approximate formulas, consider a rod of steel 7 ft. long and 3 in. in diameter (Fig. 5). In this case,

$$k_0^2 = \frac{r^2}{4} = \frac{1.5^2}{4 \times 144} = 0.00391;\ 12\,k_0^2 = 0.0469$$

which is small enough compared with $l^2 = 49$ to be neglected. For a piece of the bar $\frac{1}{2}$ ft. long, however, $12\,k_0^2$ is about 19% of l^2. The weight per foot of such a rod will be 24 lb., so that its mass per foot will be $K = \frac{24}{32.2} = 0.7455$.

Let, as before, a force of 1 000 lb. be suddenly applied 1 ft. from its lower end. Then,

$$l = 7;\ P = 1\,000;\ x = 2.5;\ e_1 = 1;\ e_2 = 6;$$

$$r = \frac{l^2}{12\,x} = \frac{49}{30} = 1.633$$

$$Q_1 = \frac{12 \times 1\,000 \times 2.5}{343}\,(1 - y)\left(4.633 + \frac{y}{2}\right) = 43.72\,(1 - y)\,(9.266 + y)$$

$$H_1 = \frac{6 \times 1\,000 \times 2.5}{343}\,(1 - y)^2\left(4.8 + \frac{y}{3}\right) = 14.58\,(1 - y)^2\,(14.4 + y)$$

$$Q_2 = \frac{12 \times 1\,000 \times 2.5}{343}\,(6 - z)\left(1.133 - \frac{z}{2}\right) = 43.72\,(6 - z)\,(2.266 - z)$$

$$H_2 = \frac{6 \times 1\,000 \times 2.5}{343}\,(6 - z)^2\left(0.133 - \frac{z}{3}\right) = 14.58\,(6 - z)^2\,(0.4 - z)$$

These curves are plotted on Fig. 5.

Certain special cases are of great interest. First, let the force be applied opposite the centroid. In this case it has already been found that,

$$Q_1 = \frac{P\,(e_1 - y)}{l}$$

and,

$$H_1 = \frac{P\,(e_1 - y)^2}{2\,l}$$

Also, Q_2 and H_2 will be the same. Taking the same data as in the last numerical case, except that,

$$e_1 = 3.5;\quad x = 0;\quad r = \infty$$

$$Q_1 = \frac{1\,000\ (3.5 - y)}{7} = 142.9\ (3.5 - y)$$

$$H_1 = \frac{1\,000\ (3.5 - y)^2}{14} = 71.4\ (3.5 - y)^2$$

These curves are plotted on Fig. 6.

Let the force, P, be applied at the extreme lower end. Then, in the same case, still using the approximate formulas,

$$x = \frac{l}{2}; \quad e_1 = 0; \quad e_2 = l; \quad r = \frac{l}{6}$$

Q_1 and H_1 disappear, and,

$$Q_2 = \frac{12\ P\ l}{2\ l^3}\ (l - z)\left(\frac{l}{6} + \frac{l}{2} - \frac{l + z}{2}\right)$$

$$= \frac{P}{l^2}(l - z)(l - 3\ z)$$

$$H_2 = \frac{6\ P\ l}{2\ l^3}\ (l - z)^2\left(\frac{l}{6} + \frac{l}{2} - \frac{2\ l + z}{3}\right)$$

$$= -\frac{P}{l^2}\ (l - z)^2\ z$$

Substituting numerical values:

$$Q_2 = 20.4\ (7 - z)\ (7 - 3\ z)$$

$$H_2 = 20.4\ (7 - z)^2\ z.$$

These curves are plotted on Fig. 7.

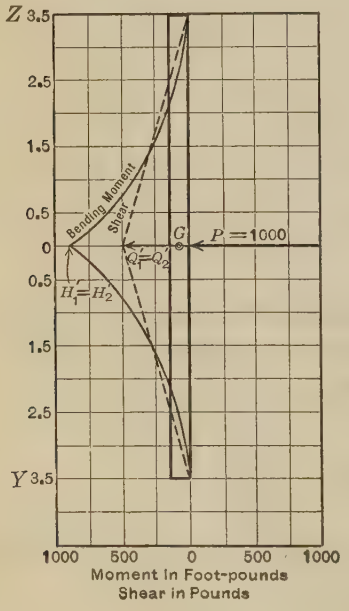

FIG. 6.

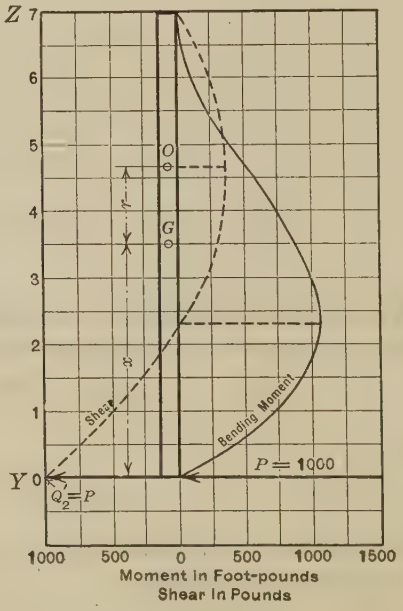

FIG. 7.

It may be noted that instead of the impressed force, P, the acceleration of the point of application may be given. Equation (12) is,

$$\frac{d\,\omega}{dt} = -\frac{12\,P\,x}{K\,l\,(l^2 + 12\,k_0^2)}$$

but,

$$(r + x)\,\frac{d\,\omega}{dt} = \frac{d\,v}{dt} = a$$

the acceleration of the point of application of P. Then,

$$P = -\frac{K\,l\,(l^2 + 12\,k_0^2)\,a}{12\,x\,(r + x)} = \frac{-\,K\,l\,(l^2 + 12\,k_0^2)\,a}{l^2 + 12\,k_0^2 + 12\,x^2}$$

If $12k_0^2$ is neglected, these become,

$$P = \frac{-\,K\,l^3\,a}{12\,x\,(r + x)} = \frac{-\,K\,l^3\,a}{l^2 + 12\,x^2}$$

In the numerical instance last cited where $12\,k_0^2$ is neglected, if the force, P, be applied at a point 1 ft. from the lower end, then,

$$a = \frac{-\,1\,000\,(49 + 12 \times 6.25)}{0.7455 \times 343} = -\,485 \text{ ft. per sec. per sec.}$$

If opposite the centroid,

$$a = \frac{-\,1\,000\,(49)}{0.7455 \times 343} = -\,191.5 \text{ ft. per sec. per sec.}$$

Since this last movement is pure translation,

$$a = -\frac{P}{m} = \frac{-\,1\,000 \times 32.2}{7 \times 24} = -\,191.5 \text{ ft. per sec. per sec.}$$

Finally, if P is applied at the lower extremity,

$$a = \frac{-\,1\,000\,(49 + 12 \times 12.27)}{0.7455 \times 343} = -\,767 \text{ ft. per sec. per sec.}$$

DISCUSSION

WILLIAM R. OSGOOD,* ASSOC. M. AM. SOC. C. E. (by letter).—The location of the center of percussion (derivation of Equation (10)) seems unnecessary, since it is commonly given in elementary textbooks on mechanics.

A simpler derivation of the equations of the shearing forces (Equations (13) and (15)) and of the equations of the bending moments (Equations (14) and (16)) occurs to the writer. According to d'Alembert's principle, the problem may be reduced to one of equilibrium if the rod is acted on by the force, P, and if at the same time each particle of the rod is acted on by a force equal and opposite to the product of the mass of the particle and its acceleration.

With the positive direction downward, the equation of motion applying in the present case gives for the rod (see Fig. 8), $-P = Kla_g$, in which, a_g is the acceleration of the center of mass of the rod; also, with counterclockwise rotation positive, $a_g = r\alpha$, in which, α is the angular acceleration of the rod. As the acceleration of any point of the axis of the rod is proportional to the distance of the point from O, the free axis of rotation, the acceleration, a, of any point of the axis a distance, y, to the left of P, or z to the right of P, becomes,

$$a = \frac{r + x \begin{cases} + y \\ - z \end{cases}}{r} a_g$$

$$= -\frac{P}{K l r}\left(r + x \begin{cases} + y \\ - z \end{cases}\right) \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

also,

$$\alpha = -\frac{P}{K l r} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The equal and opposite forces which, according to d'Alembert's principle, are to act on the particles of an elementary slice of the rod taken between two adjacent cross-sections, may be replaced by the force, $-adm = -aKdl$, and the couple, $-\alpha dI = -\alpha k_0^2 K dl$. The loads to be applied to the rod are, then, those shown in Fig. 8, namely, the force, P; a uniformly varying load, $-aK$, per unit of length; and a uniform distribution of couples, $-\alpha k_0^2 K$, per unit of length.

The shearing force at any section a distance, y, to the left of P is, with the usual convention for positive and negative shears,

$$Q_1 = -\frac{(-a_1 - a)K}{2}(e_1 - y)$$

in which, a_1 is the value of a at the left end of the rod, that is, the value for $y = e_1$. By substituting a and a_1 obtained from Equation (27) and reducing, Q_1 becomes,

$$Q_1 = -\frac{P}{l r}(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right)$$

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The bending moment at the same section is, with the usual convention for positive and negative moments,

$$H_1 = -\frac{-a\,K\,(e_1 - y)^2}{2} - (-a_1 + a)\,K\,\frac{e_1 - y}{2} \times \frac{2\,(e_1 - y)}{3} + \alpha\,k_0^2\,K\,(e_1 - y)$$

By substituting a and a_1 obtained from Equation (27) and α from Equation (28) and reducing, H_1 becomes,

$$H_1 = -\frac{P}{2\,l\,r}\,(e_1 - y)^2\left(r + x + \frac{2\,e_1 + y}{3} + \frac{2\,k_0^2}{e_1 - y}\right)$$

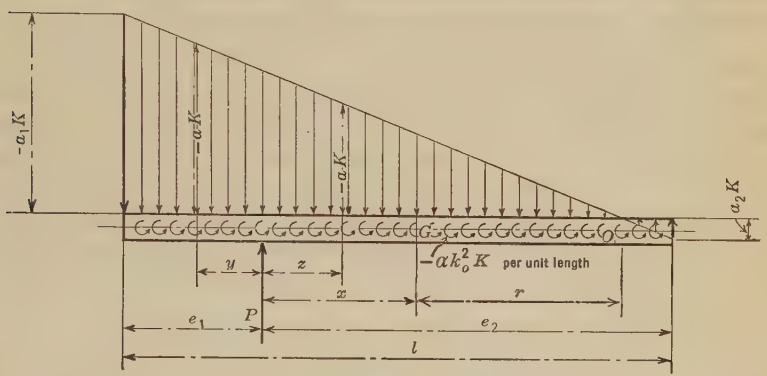

FIG. 8.

The shearing force at any section a distance, z, to the right of P is,

$$Q_2 = -\frac{(a_2 + a)\,K}{2}\,(e_2 - z)$$

in which, a_2 is the value of a at the right end of the rod, that is, the value for $z = e_2$. By substituting a and a_2 obtained from Equation (27) and reducing, Q_2 becomes,

$$Q_2 = \frac{P}{l\,r}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right)$$

The bending moment at the same section is,

$$H_2 = \frac{a\,K\,(e_2 - z)^2}{2} + (a_2 - a)\,K\,\frac{e_2 - z}{2} \times \frac{2\,(e_2 - z)}{3} - \alpha\,k_0^2\,K\,(e_2 - z)$$

By substituting a and a_2 obtained from Equation (27) and α from Equation (28) and reducing, H_2 becomes,

$$H_2 = -\frac{P}{2\,l\,r}\,(e_2 - z)^2\left(r + x - \frac{2\,e_2 + z}{3} - \frac{2\,k_0^2}{e_2 - z}\right)$$

By substituting for r in the denominator of the right-hand side of the equations for Q_1, H_1, Q_2, and H_2, the value given by the author's Equation (11), the equations just derived for Q_1 and Q_2 become the same as the author's

Equations (13) and (15), except that the sign of Q_1 will be opposite, and the equations here given for H_1 and H_2 obtain a form to which the author's Equations (14) and (16) can easily be reduced with opposite signs. The opposite signs in the three cases mentioned are due to the fact that the usual convention for the signs of shearing forces and bending moments has been followed throughout in this discussion.

The following brief bibliography may be of interest in connection with this paper:

Martin, L. A., Jr. Textbook of Mechanics. N. Y., 1912, p. 200.
Zschetzsche. *Zeitschrift* des Vereines deutscher Ingenieure, 1894, p. 134.
Flamant, A. Résistance des Matériaux. Paris, 1897, p. 635.
Ostenfeld, A. Teknisk Elasticitetslære. Copenhagen, 1924, p. 293.

Henry D. Dewell,* M. Am. Soc. C. E. (by letter).—The interest of engineers in the subject of the stresses set up in engineering structures by earthquakes is increasing. In particular, the study of the subject by American engineers may be said to have begun after the San Francisco earthquake of 1906. This interest gradually waned in the years following 1906, but was again revived after the great Japanese earthquake in 1923, and has been growing since, the subsequent St. Lawrence, Santa Barbara, and Montana earthquakes having shown that no section of the country can truthfully be said to be immune.

The Society has a Special Committee which is diligently studying the subject of earthquakes with particular reference to their effect on engineering structures. Heretofore, and even at the present time, it is the Japanese engineers who have given most thought to the subject, and the results of their studies and conclusions have been published from time to time. Unfortunately, but a small part of this material is available in the English language. At least one treatise on the design of modern buildings to resist earthquakes has been published by Japanese engineers,† which has been translated by the Committee. The writer knows of no book in the English language that is comparable to this work. The Committee has also completed the translation of a mass of pertinent data furnished by its Japanese members. In addition, it is also engaged in experimental work, having undertaken the determination, by accurate actual measurement, of the periods of natural elastic vibration of a number of buildings and other structures in San Francisco and vicinity. It is pertinent also to point out that the California Institute of Technology and Leland Stanford, Jr., University have undertaken to determine the effect of vibratory motion on structures, by means of a shaking table and models of engineering structures.

This résumé is given to emphasize what is true, but not often realized, namely, that the intense and systematic study of the causes and effects of the great natural phenomenon of earthquakes is of recent date. There is a good deal about the nature, cause, and effect of earthquakes that is not known, and there probably are phases that may never fully be understood. Nevertheless,

* Cons. Engr., San Francisco, Calif.
† Treatise by Dr. T. Naito.

experience and study have shown that earthquake-proof structures are not only possible, but are economically possible.

The general subject of the resistance of structures to earthquake shock may be divided into two major parts. The first is the study of the earthquake itself—its cause, the manner in which the shock is propagated through the earth's crust, and the characteristics of the earthquake wave, including period, amplitude, and direction, and consequent intensity at various distances from the origin. These points are primarily the object of the seismologist's study. The results are, obviously, of vital interest to the structural engineer. Indeed, without the seismologist's conclusions the engineer could do little, and without a basic knowledge of seismology he is at a great disadvantage in attempting to design for safety against an earthquake.

The second major portion of the general subject is the finding of the stresses set up in an engineering structure by the earthquake waves, together with the proper design of the framework to resist those stresses. This division of the subject falls fundamentally in the province of the engineer, and the seismologist very properly turns over this phase of the problem to him.

The engineer is primarily interested in the effects of that class of earthquakes known as "tectonic", as distinguished from those caused by volcanoes. The tectonic earthquake is believed to be caused by a slipping along a "fault plane" of one block of the earth's crust upon another. The slip and resultant "elastic rebound" are held to send out elastic waves throughout the materials of which the earth is composed. These elastic waves, as registered at any point of the earth's surface, are complex in nature. In general, however, two types are to be distinguished, namely, longitudinal and transverse. The longitudinal waves, similar in character to sound waves in that the vibrations are in the direction of propagation, are those of compreession and dilatation. The transverse waves, on the other hand, are akin to light waves, in that the vibrations are transverse to the direction of propagation.

The effect of passing through or impinging on media of varying densities in the earth's crust is to cause refraction and reflection. These phenomena, together with the interference of the waves, result in the complexity of the waves noted in the record of the seismograph.

The first earthquake waves to be recorded at any point of observation are the longitudinal waves; later, the transverse waves are recorded. Thus, it has been a relatively common experience in earthquakes to note that the shock came first from one direction, and then from a second, the two directions being frequently at right angles to each other. Also, if the focus of the earthquake was near the observer, the first effect felt may have been a sudden "shove" or "bump", followed by a rapid swinging motion. Students of seismology are not in agreement on the importance to engineering structures of the effect of the first longitudinal wave. Aside from seismographic records, which seem to be contradictory on this point, the testimony of many observers gives evidence leading to the belief that very often the collapse of some structure is coincident with the arrival of this first shock, which strikes the foundation of the building as with a blow. It is to this phase of the subject that the paper is believed to apply.

As noted in the Synopsis, Omori, the late and eminent Japanese seismologist, observed that very rarely did tall chimneys fracture at the base. He is credited with the statement that tall free standing chimneys will always tend to break at a point about two-thirds of their height. An examination of the records of such fractured chimneys, from a study of which his conclusions were drawn, will show that, although the average location of such fractures is at a point approximately two-thirds of the height of the chimney, the variation from such average is large.

Again, as far as the writer has been able to determine, the method of computation used to find the stresses at any section of the chimney is to assume that these stresses result from a force acting at the center of gravity of that part of the chimney above the section under consideration, and equal in amount to the mass of that portion multiplied by the assumed value of the acceleration of the earthquake wave. Expressed mathematically,

$$m = \frac{W}{g} \quad \ldots\ldots (29)$$

$$F = ma = \frac{W}{g} a \quad \ldots\ldots (30)$$

$$M = Fx = \frac{W}{g} ax \quad \ldots\ldots (31)$$

in which,

m = mass of chimney above the horizontal cross-section under consideration.
W = weight of this mass.
x = vertical distance from this horizontal cross-section to the center of gravity of the mass above.
g = acceleration of gravity.
a = horizontal acceleration of earthquake.
F = force of earthquake acting on the portion of the chimney above the horizontal section.
M = effective moment of the force of the earthquake on the horizontal section of the chimney.

Now, with the dimensions of masonry chimneys as ordinarily constructed, it may be easily shown that the weakest section of the chimney as against the bending moment, M, is at the base. In other words, if the action of the forces on the chimney is as here indicated, which has heretofore been assumed,* then all brick chimneys should fracture at the base, which conclusion is not in accord with the facts. The use of a higher acceleration in computing the bending moments at about two-thirds of the height than is used for the base, as advocated by some engineers, is certainly not a logical method of reconciling theory with observation. Evidently, this view of distribution of shears and bending moments in tall slender structures, such as chimneys, is incorrect.

Again, if the computed values of the varying section moduli of the sections of chimney from top to base be plotted to scale, that is, the vertical distances

* For example, "Building Structures in Earthquake Countries," by Alfredo Montel; "Reinforced Concrete and Masonry Structures," by Hoole and Kinne, pp. 599, 600, 665.

of successive cross-sections above the base being laid out as ordinates, and the corresponding section moduli be laid out properly as abscissas, and, further, if the curve of the theoretical bending moment as found by Professor Le Conte (Fig. 7) be superimposed on the first curve (to the same vertical scale), it will be seen that the variation of the two graphs is roughly the same for the upper two-thirds portion of the chimney. Hence, after making allowances for possible sharp changes in section of chimney, instead of a uniform rate of change in section modulus from top to bottom, for the effect of unequal quality of workmanship in construction and for the known uncertain value of brick masonry in tension, the records of brick chimneys fractured by earthquakes are in accord with the distribution of shears and bending moment as deduced by the author.

The writer realizes what Professor Le Conte has pointed out, that the paper deals with a purely theoretical problem, and that the fundamental assumptions of rigidity and freedom from restraint are not realized in any engineering structure. Nevertheless, he believes that the formulas given in the paper are roughly applicable to the solution of the probable stresses due to earthquakes in certain engineering structures.

The effect of oscillation on a structure has not been covered by the author. However, he and Professor Hoskins, of Leland Stanford, Jr., University, at the suggestion of the Society's Special Committee on the Effect of Earthquakes on Engineering Structures, have investigated this problem and have solved it for the simple and ideal case of a vertical beam subjected to an oscillating force, producing simply harmonic motion of the base. The results thus proved mathematically have been demonstrated experimentally by Professor Le Conte. The resulting conclusions are remarkably simple of practical application. The determining feature is the ratio of the natural period of elastic vibration of the structure to the period of the oscillating force. Also, Dr. Naito, a Japanese engineer, has shown by a survey of buildings injured in the Tokyo earthquake of 1923, that the damage to such buildings is apparently dependent on the ratio of their natural periods of elastic vibration to the average period of vibration of the most violent part of the shock. These further investigations are being studied by the Committee.

The profession should be grateful to Professor Le Conte for the results of his investigation. The writer hopes the paper will arouse interest in the general subject of the earthquake resistance of engineering structures to the end that intensive study of the subject may become more general, and that eventually certain fundamental laws may be evaluated. Until engineers, in general, are in agreement on the proper design of structures to resist earthquakes, but little progress can be expected in the matter of influencing the prospective owners of buildings and other structures to consider possible earthquake effects when planning the structures.

A. C. Schwager,* Esq. (by letter).—The author has investigated the stresses in an entirely rigid bar, having no support whatsoever. The importance of the problem will justify a comparison between this bar and an elastic

* San Francisco, Calif.

bar, as found in actual structures. It is of particular interest to determine if any of the main characteristics of a rigid bar—the center of percussion as well as the mode of distribution of moments and shears—finds its analogy in the case of an elastic bar. To keep this comparison within the form of a discussion, only the most important equations are reproduced and the moment and shear charts are given as to relative values only.

An elastic rod, excited by an impulse, will finally assume a motion which can be determined as follows:* Referring to Fig. 9, y denotes the deflection of the bar from its original position at the distance, x, from the lower end. If, furthermore, k = radius of gyration, and $b = \frac{E}{\rho} = \frac{\text{modulus of elasticity}}{\text{volume density}}$, the differential equation for the movement is:

$$\frac{d^2 y}{d t^2} + k^2 b^2 \frac{d^4 y}{d x^4} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

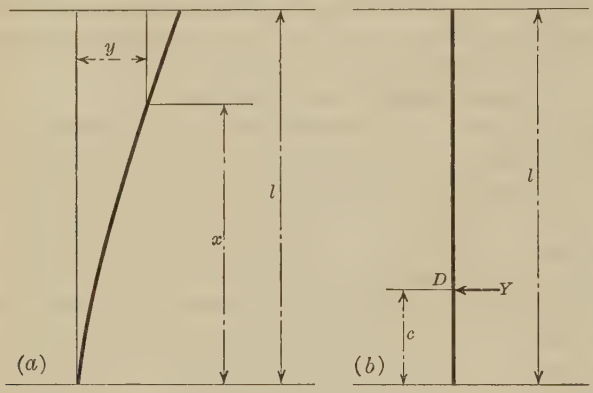

FIG. 9.

If both ends are assumed to be free:

$$y = \Sigma_r P_r u_r (x) \cos \left(\frac{k b}{l^2} m_r^2 t + e \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

in which,

$$u_r (x) = (\cos m_r - \cosh m_r) \left(\cos \frac{m_r x}{l} + \cosh \frac{m_r x}{l} \right) + (\sin m_r + \sinh m_r) \left(\sin \frac{m_r x}{l} + \sinh \frac{m_r x}{l} \right) \quad \ldots\ldots\ldots\ldots (34)$$

and the values of m satisfy the equation

$$\cos m_r \cosh m_r = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

There are an infinite number of values for m, the lowest ones being:

$$m_1 = 4.73$$
$$m_2 = 7.85$$
$$m_3 = 11.00$$
$$m_4 = 14.14$$
$$m_5 = 17.28$$

* The fundamental equations for the movement of an elastic bar are taken from Rayleigh's "Theory of Sound."

The curves, $u_1(x)$, $u_2(x)$, and $u_3(x)$, representing the component vibrations, are shown in Fig. 10 (a), (b), and (c). Their frequency is given by,

$$f_r = \frac{k\,b}{2\,\pi\,l^2}\,m_r^2 \qquad (36)$$

Furthermore, the moment and shear curves are plotted, being proportional to the second and third derivative, respectively, of Equation (34). The points marked A in Fig. 10 do not move at all, considering one particular vibration only and disregarding constant translation and rotation of the bar. The distribution of the moments in all cases is symmetrical with the middle of the bar and independent of the point of application of the disturbing impulse. Fig. 10 (a) which is a modification of Fig. 10 (b) shows the lowest vibration of a bar supported at its lower end; in the distribution of moments it is closely similar, although not identical, to that shown in the author's Fig. 7.

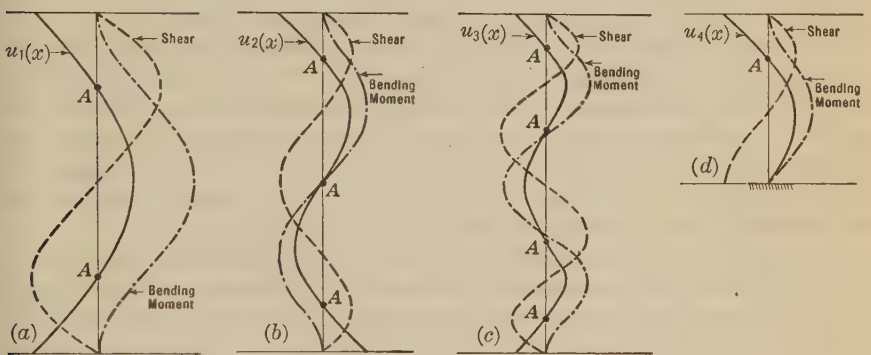

FIG. 10.

The relative contribution of the different vibrations to the actual movement of the bar is given by the terminal conditions, which determine the values, P_r, in Equation (33). Assuming the bar to be excited by an impulse of a momentum, Y, at the distance, c, from its lower end (Fig. 9 (b)) and denoting by ω the area of the cross-section:

$$y = \frac{l^2\,Y}{k\,b\,\rho\,\omega}\left(\frac{(u_1(c)\,u_1(x)}{m_1^2\int u_1^2\,d\,x}\sin\left(\frac{k\,b}{l^2}\,m_1^2\,t\right) + \ldots\right.$$

$$\left. + \frac{u_r(c)\,u_r(x)}{m^2\int u_r^2\,dx}\sin\left(\frac{k\,b}{l^2}\,m_r^2\,t\right) + \ldots\right) \qquad (37)$$

Thus, the motion of the bar is entirely determined and composed of an infinite number of fundamental vibrations. Actually, the impulse occurs in the entire neighborhood of Point D, which will eliminate the excitation of the vibrations of higher frequencies, a fair approximation for the moments thus being obtained by considering only the lowest vibrations. Maximum moments and shears can be obtained by adding the respective absolute values pertaining to the fundamental vibrations, as determined in Fig. 10.

The comparison between rigid and elastic bars shows that positive as well as negative moments appear in the latter, a result which is contrary to that of the rigid bar. The conception of the center of percussion can be found not only in the case of a rigid bar, but in a modified manner also in the fundamental vibrations of an elastic bar.

The author's statement that "the elasticity * * * greatly reduces the magnitude of the stress" holds true in general. Although no qualitative comparison has been made, it seems possible that the stresses at the upper end of an elastic bar, vibrating with a large number of fundamental components, may exceed those occurring in a rigid bar, both being excited by an impulse of the same magnitude at the lower end.

Considering Equation (36) for the frequency, the question of possible resonance with a periodic force arises. This case is of considerable importance and has been discussed in other publications*.

Joseph N. Le Conte,† Esq. (by letter).—It is interesting to note that Mr. Dewell finds a fair agreement between the points of maximum stress in the free prismatic rod problem, and the points of failure of certain tall chimneys which have been subjected to earthquakes. It would appear that this agreement will become more nearly parallel, the more rigid the structure, and the more freely the structure is allowed to rock on its base. If the rod or structure is perfectly rigid, but fixed at the base, so that the initial tangent is vertical, then the accelerations of all points will be the same as that of the base, and Mr. Dewell's Equations (29) to (31) will hold accurately.

The effect of elasticity in this last case (of fixed base) is not difficult to take into account if the acceleration of the base is constant. Of course, such a type of motion could not persist for any considerable time, and would not exist in an earthquake disturbance, but the first phase of the motion might follow such a law.

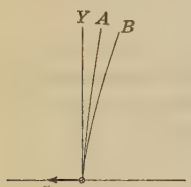

Fig. 11.

Imagine an elastic rod in a vertical position initially, and then imagine a constant acceleration, a, given to the base. It can be shown that the rod will vibrate back and forth with a simple harmonic motion relative to the moving base, O (Fig. 11), about a mean elastic curve, $O\ A$, and with a period equal to the natural period of the rod. In other words, it will swing from the position, $O\ Y$ (straight), to the position, $O\ B$ (curved), with a simple harmonic motion, $O\ A$ (approximately half way between), being the curve from which amplitudes are measured. Furthermore, the curve, $O\ A$, is the same as the elastic curve of a uniformly loaded horizontal beam fixed at one end and at rest, the uniform rate of loading being the mass per unit of length multiplied by the acceleration of the base. The maximum stresses would occur in the position, $O\ B$, which would be twice those of the position, $O\ A$.

It is also interesting to observe the methods used by Professor Osgood in obtaining the equations of bending moments and shear in the rigid rod. In regard to his criticism of the writer's derivation of the formula for the center of percussion, the writer did this merely for the sake of completeness.

* *Zeitschrift für angewandte Mathematik und Mechanik*, 1923, Vol. 3.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1652

TESTS OF MANHOLE COVERS*

By T. J. Corwin, Jr.,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. M. M. O'Shaughnessy and T. J. Corwin, Jr.

Synopsis

This paper is a résumé of a report by the Division of Civil Engineering of the Pacific Gas and Electric Company, in which the theoretic stresses and deflections of manhole covers are compared with the values obtained by test.

A series of tests was made to determine the merits of the different designs and also to determine if a saving could be effected by using cast steel instead of cast iron.

This investigation was brought about by a request of the Division of Electric and Steam Distribution of the Pacific Gas and Electric Company for a change in the design of the manhole covers that would result in a saving in the initial cost and in an increased strength to conform with present traffic requirements.

Introduction

The only information regarding previous tests on manhole covers was obtained through engineering magazines. In one test the ultimate strengths of three types of covers were obtained, but no attempt had been made to determine the stresses in the covers. The other test based the strength of the cover on its ability to withstand a suddenly applied load. The magnitude of the force depended on the height from which the load was dropped.

* Published in January, 1927, *Proceedings*.

† Asst. Engr., Pacific Gas & Elec. Co., San Francisco, Calif.

The strength of the covers used in the systems of the Pacific Gas and Electric Company had never been determined by test. However, at present, replacement of the cast-iron covers due to breakage is exceedingly small and breakage occurs only under unusual traffic conditions. In the past, when a cast-iron cover failed under severe traffic conditions in the East Bay District of the Company, the cover was replaced by one of cast steel of the same design. The cast-steel covers have been satisfactory, so it can be assumed that their ultimate strength is sufficient for existing traffic conditions. The cast-steel ventilating cover has been subjected to the most severe traffic conditions, but no failures have been reported.

Analysis of Design of Old Covers

After studying the design of the old ventilating cover, Fig. 1 (No. 1-1149), it appeared evident that a reduction could be made in the weight, for the two systems of ribs were not of equal strength. These covers were of cast steel and weighed 630 lb. The ribs were of constant thickness for the full width of the cover, whereas a varying thickness could be used in order to reduce the weight.

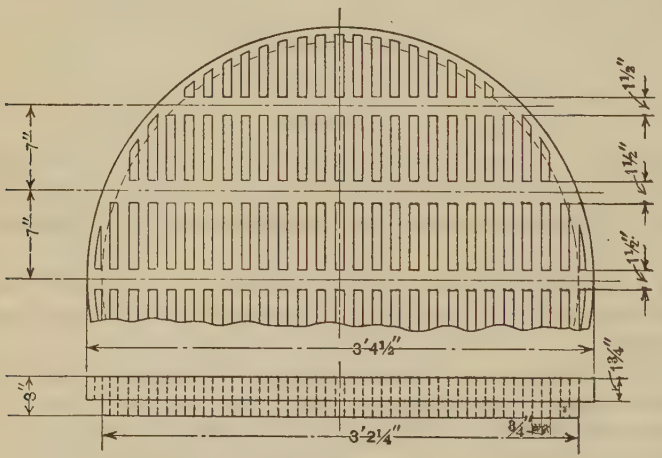

Fig. 1.—Old Ventilating Cover (No. 1–1149).

A study of the solid cover, Fig. 2 (No. 31086), did not indicate a surplus of material and unless radial ribs were detrimental to the strength, no reduction in the weight could be made, assuming a flat plate would support no greater load than theoretical computations indicated.

Re-Design of Covers

A load of 15 000 lb. plus an impact factor of one-third was assumed as the working load. On this basis the covers were designed for an ultimate load of 70 000 lb. In order to simplify the theoretical computations this load was applied through a cylindrical loading block instead of in the manner it would be applied by the wheel of a truck.

The weight of the cover was a factor that was kept constantly in mind. However, the minimum thickness of casting advisable was the factor that really established the final weight, after the strength requirements had been satisfied.

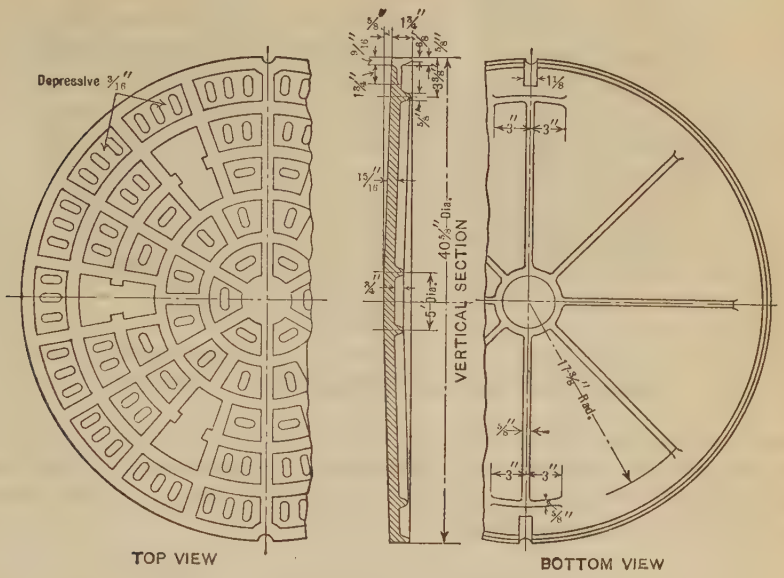

FIG. 2.—OLD SOLID COVER (No. 31086).

Ventilating Cover (*Fig. 3*).—The re-design of the ventilating cover (No. 25370) followed two requirements shown by the old cover: The area of the openings and the height of the supporting ring. The main ribs were spaced so that the design load would be carried by three ribs. From the symmetry of the design one-half the load will be taken by each set of ribs. The spans of the three center ribs for the purpose of analysis were taken as equal. The ribs were made equal in area. To compute the bending moment the load was considered as concentrated at a point equal to the average radius of the loading block, from the center of the cover. The small grate bars were of sufficient strength to transmit the load to the main ribs.

Solid Cover (*Fig. 4*).—Assuming that the radial ribs were not the most efficient type of reinforcing, only two other systems of ribs could be used. One system was in the form of concentric circular ribs, the other a system of rectangular ribs similar to those used in the ventilating cover.

The re-design of this cover (No. 36863) required considerable study. With the tensile strength of the metal less than one-half its compressive strength, it appeared evident that all ribs should be on the top or compressive side for a cover of minimum weight and maximum strength. Deep ribs on the top would be objectionable to traffic and pedestrians. The cost of filling in between the ribs with asphalt or cement would probably more

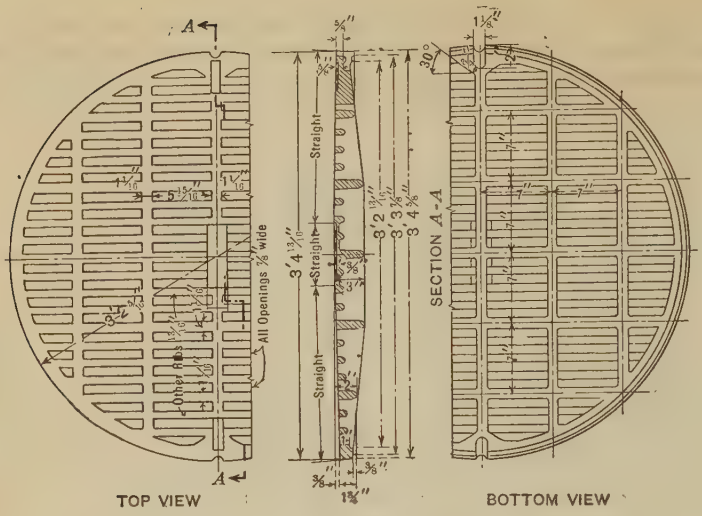

FIG. 3.—NEW VENTILATING COVER (No. 25370), CAST STEEL.

than offset the saving in weight of metal. The cover could not be domed for it would be objectionable to traffic, and a slight crown will not increase the strength. Placing ribs on the bottom would seem to defeat the purpose in view when adding them.

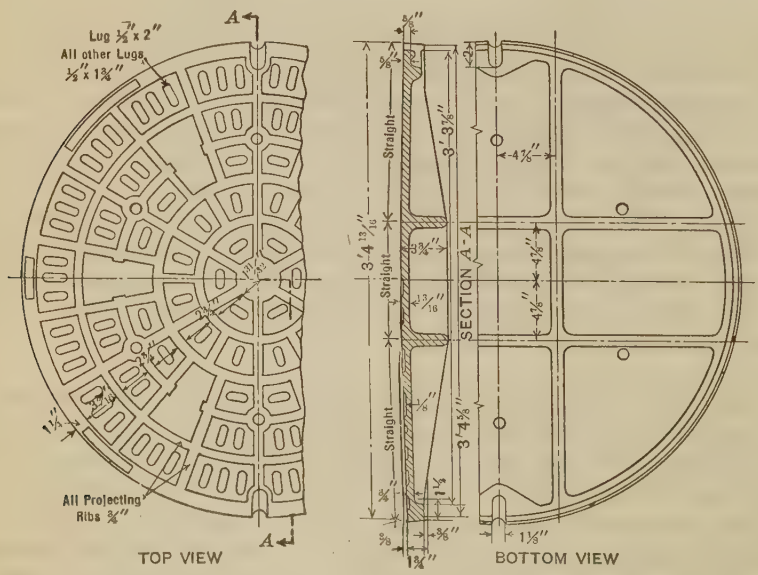

FIG. 4.—NEW SOLID COVER (No. 36863).

According to the theory of flexure, the fiber stress is proportional to the distance from the neutral axis. The distance to the extreme fibers in tension will be two to three times the distance to the extreme fibers in compression.

This does not seem to be an economic design. However, a cover was designed with the plate thickness the same as the old design, and four ribs were added on the bottom, two sets of two at right angles.

Another solid cover (Cover *B*, Fig. 5) was designed with the ribs in the form of concentric circles (See Table 4, Record "B"). This design was based on the assumption that contraction occurred on radial lines and the stresses might be equalized in this manner.

Procedure for Test and Equipment

A 300 000-lb. wheel-press was used for the test. To insure an equal distribution of the load a special casting was designed to transmit the load from the manhole frame to the frame of the wheel-press.

The tests were made under conditions approximating those that will be obtained in the field on all new installations. The lower side of the cover and its seat in the frame were machined to insure a uniform bearing. This will prevent the rattling of the cover in the frame and also some breakage due to unequal bearing. The lower side of the manhole frame and the surface of the bearing plate were machined so that the cover would be normal to the ram of the press.

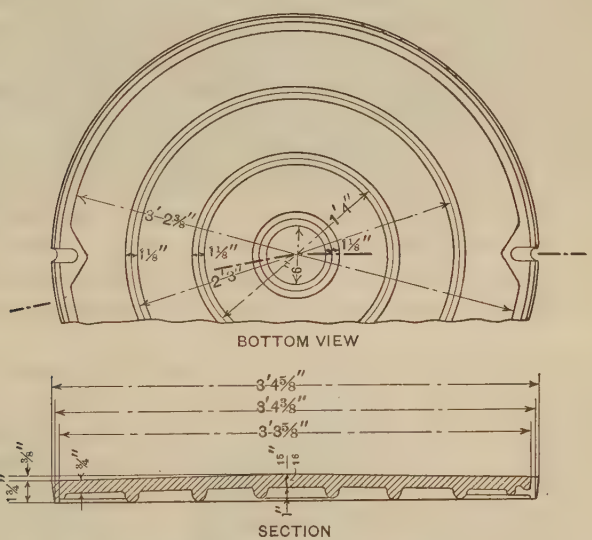

Fig. 5.—Solid Ribbed Cover.

The extensometers were made specially for the test. The main consideration was to obtain an extensometer that could be clamped to the ribs of the cover. Two Ames dials indicating the deformations to the nearest 0.001 in. were used with each extensometer. The extreme fiber deformation is readily computed from the deformations indicated by the two dials and their respective distances from the extreme fiber. The extensometer rods passed through holes in the manhole frame, placing the dials on the outside of

the frame so that the deformation indicated on the dials could be easily recorded.

The center deflection of the cover with respect to the frame of the wheel-press was determined with an Ames dial. This method included the deflection of the bearing plate. Considering the relative moments of inertia of the two sections, it is apparent that the deflection of the loading block is negligible compared with the center deflection of the cover.

A 6-in. gauge length was used for all but one test. The loading block was designed so that the bending moment would be constant for the center 7 in. of the cover. Fig. 6 shows the complete equipment.

Two extensometers were connected at right angles on the covers. From the symmetry of the design and the fact that all surfaces were machined, one would assume that the deformations at right angles would be equal. This assumption was verified in the tests.

The pump for the hydraulic press was not equipped with an accumulator, and, therefore, it was difficult to obtain a steady increase in the pressure. In many cases, it was necessary to reduce the load a few thousand pounds in order to obtain the desired value. This was particularly true at the lower load intensities, for the pressure built up rapidly.

The smallest practical increment of load was one division on the 3 000-lb. Ashton pressure gauge. This was equal to a 3 180-lb. increment of load on the cover. The center deflection and the fiber deformations were recorded simultaneously. The dials were removed when the elastic limit had been passed and the cover tested to failure.

Zero readings taken on the extensometer were apparently in error and not proportional to the deformation expected for the first increment of load. The zero points were determined by extrapolating the load-deformation curves. With the short gauge length and the relatively low elastic limit of cast steel the percentage of error between the theoretical stress and the measured value might be large and still be within the practical limit of the equipment.

The center deflection was recorded for practically all covers, but not with the intention of computing the stress or effective moment of inertia. It was thought that the elastic limit might be indicated by a sudden increase in the rate of the deflection, or it might serve as a basis of comparison for the different types of covers. The modulus of elasticity of the metals used for the covers was determined by tension tests. The value for cast steel was in excess of the value normally used. The value for "semi-steel" was approximately the same as that normally used for cast iron.

Comparison of Theoretical and Measured Stresses

Ventilating Cover (Fig. 3).—The bending moment and the section modulus were computed and the corresponding stress was determined. The unit stress was also computed from the deflection data obtained during the test. The figures in Table 1 show the variation in the stresses. One would expect the theoretical stress to be higher than the measured stress, for the

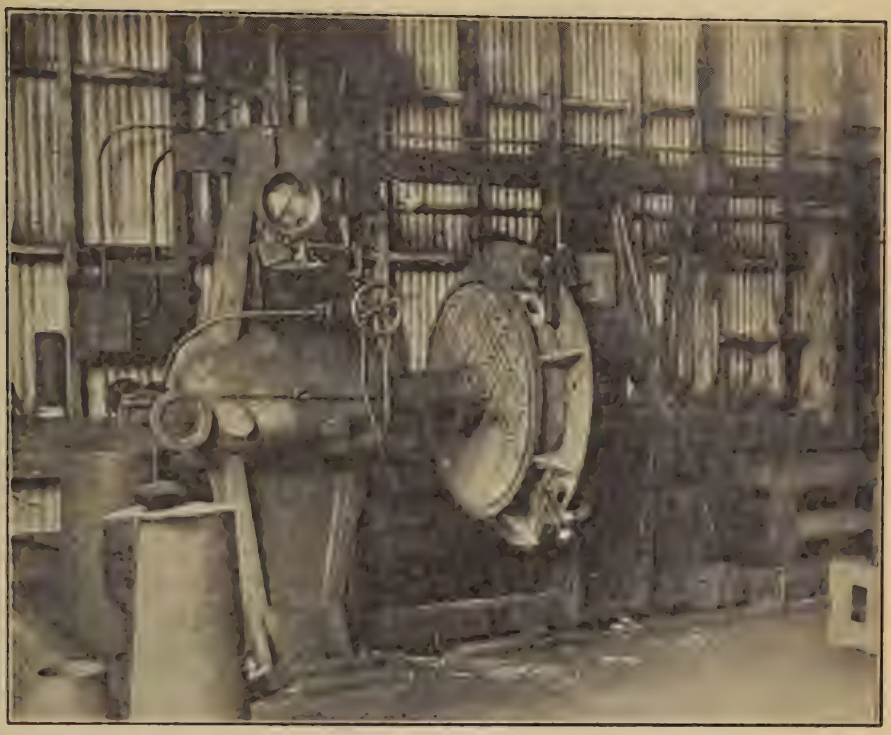

Fig. 6.—Apparatus for Testing Manhole Covers.

Fig. 7.—Typical Fractures of Manhole Covers.

ribs on the sides of the center ribs must also contribute to the strength of the cover. The difference in these stresses should become greater as the load increases, for the deflection of the center ribs will cause the side ribs to carry a larger part of the load. Some variation is also due to the fact that the moment of inertia was assumed as constant in the computations. The ribs have a variable moment of inertia, but as the moment of inertia does not vary directly with relation to the span length, the exact formula was not used. The true stresses should fall between these assumed conditions. The deflection is greater with this section than with ribs of constant section, therefore the stress computed from the measured deflection is probably 10 to 20% higher than the actual value.

Cover *A* (Fig. 3), with an exceptionally low elastic limit, did not fail until the load was in excess of 100 000 lb. At this load it had dished in about 1½ in. When the load was released the cover had a permanent set of 1 in. The only indication of fracture was at the junction of the main ribs and the supporting ring.

Cover No. 11 was tested to 69 000 lb. At this load the deformation of the extreme fibers was increasing rapidly. The cover had a permanent set of ⅝ in., indicating that the ultimate load was not greatly in excess of the maximum test load.

Test bars cut from Cover *A* showed that the ribs contained many flaws. With a casting of this type it would be difficult to obtain metal entirely free from flaws. The yield points determined by testing the cover, by tension tests, and as specified by the steel foundry, all agree satisfactorily.

Table 1 shows a comparison of the stress computed from the theoretical moment of inertia, the effective moment of inertia as computed from the center deflection, and the values measured on the two sets of ribs.

TABLE 1.—Steel Cover, Ventilating Type, Test No. 11.

Load, in pounds.	Moment, in inch-pounds.	Unit Stresses, in Pounds per Square Inch.			
		From theoretical *I*.	From center deflection.	Ribs.	
				Horizontal.	Vertical.
9 500	72 400	10 000	12 300	8 750	10 000
15 860	121 000	16 700	20 200	13 750	15 000
22 200	169 000	23 400	26 600	17 500	20 000
28 580	217 500	29 600	32 200	23 750	25 000
34 310	261 500	35 400	38 700	25 000	30 000
40 350	307 500	41 800	45 000	35 000	40 000
46 710	356 500	48 500	52 400	43 750	45 000
53 400	407 000	.....	.....	.....	.....
60 070	458 000	.....	.....	.....	.....
63 570	485 000	.....	.....	.....	.....
66 750	509 500	.....	.....	.....	.....

Solid Covers.—At the outset, trial computations indicated that empirical formulas would not give logical values. Three analyses were used in computing the stresses. The results obtained by two analyses derived by different methods are tabulated with the measured values.

Cast-Steel Covers

These covers, Fig. 2 (No. 31086) were taken from service in the East Bay District. The extensometers could not be attached in the regular 7-in. position for a constant bending moment, so a 9-in. gauge length was used, which undoubtedly affects the accuracy of the results.

The measured values are apparently in error for the first part of the test; however, at the elastic limit the measured stresses are approximately the values expected for cast steel and more nearly agree with the theoretical computations. The results of the test on Cover No. 16 are shown in Table 2. It will be noted that for the conditions of this test (applicable only to this design) the analysis by W. H. Burr, M. Am. Soc. C. E., gives stress intensities equal numerically to the load, in pounds.

TABLE 2.—Steel Cover, Solid Type, Test No. 16.

Load, in pounds.	Center Deflection, in Thousandths of an Inch.		Unit Stresses, in Pounds per Square Inch.		
	Measured.	Computed.	Fuller and Johnson computations.*	Burr computations.†	Measured.
6 320	5	60	8 900	6 320	3 260
9 600	59	91	13 550	9 600	3 400
12 680	72	121	17 900	12 680	6 860
16 500	97	157	23 250	16 500	20 400
19 040	106	182	26 850	19 040	20 500
22 220	125	212	31 300	22 220	27 400
25 400	146	242	35 800	25 400	30 900
27 950	172	266	39 400	27 950	37 900
31 130	192	297	43 800	31 130	34 900
34 310	214	327	48 400	34 310	44 900
37 490	245	357	52 900	37 490	44 900
40 350	253	374	56 800	40 350	48 500
43 530	...	...	61 300	43 530	62 500
46 710	...	...	66 000	46 710	62 500
49 900	...	...	.....	49 900	59 900
53 400	...	...	.....	53 400	63 100
56 890	...	...	.....	56 890	56 500
60 000	...	...	.....	60 000	54 900

* "Applied Mechanics," Vol. II.

† "The Elasticity and Resistance of Materials of Engineering," 7th Edition.

"Semi-Steel" New Standard Covers (Fig. 4).

1.—The results obtained by test agree fairly well with the theoretical values; the maximum fiber stress in tension agrees with the ultimate tensile strength of the metal. It was very difficult to obtain the fiber deformation with the equipment at hand and the tests had to be made a number of times before the results proved satisfactory.

2.—*Concentric Circular Ribs.*—As the covers did not meet the requirements for ultimate strength, no attempt was made to check the stresses in the plate.

3.—*Old Standard.*—The ultimate strength was the only value desired for this cover, Fig. 2 (No. 31086).

Table 3 shows the comparison of the stresses for the cover shown on Fig. 4.

TABLE 3.—Semi-Steel Cover, Solid Type, Test No. 12.

Gauge, in pounds per square inch.	Load, in pounds.	Moment, in inch-pounds.	Unit Stresses, in Pounds per Square Inch.		
			Fuller and Johnson computations.	Burr computations.	Measured.
0	0	0	0	0	0
165	9 500	48 400	3 250	2 810	2 800
265	15 860	81 000	5 480	4 710	4 060
360	21 900	111 800	7 500	6 500	5 310
460	28 260	144 500	9 660	8 400	5 310
555	34 310	175 000	11 750	10 150	7 510
650	40 350	205 000	13 800	11 900	10 600
750	46 720	288 000	16 000	13 750	11 900
855	53 400	272 000	18 250	15 800	13 150

Discussion of Results of Test

With the design adopted by the Pacific Gas and Electric Company for the new standard, failure occurred either along one of the ribs or at an intersection of the ribs. The intersections of the ribs were readily located by depressions in the top of the cover. After failure had occurred, inspection showed that the properties of the metal at the intersection were different from those of the main mass; the change in color was marked. All covers failed in practically the same manner. Considering the relative weakness of the ribs in tension, one would expect the cover to fail across the diameter at right angles to a set of ribs. This was almost the case, except for the apparent weakness at the intersection of the ribs, for in each cover failure occurred through at least two of these points. The first cover just reached the design strength required. Other tests indicated that the cover was capable of resisting an ultimate load of 100 000 lb. and that failure at 69 000 lb. was due to a flaw in the ribs of the specimen.

The cover with concentric ribs failed principally on radial lines, except for encircling the small center rib. With the old standard cover, failure occurred almost without exception adjacent to a rib of the cover. The first crack appeared along a rib and the rapid deflection that followed was probably the reason for failure occurring at such a small load. Fig. 7 shows the typical failures for the three different designs.

A discussion of the distribution of stress in a circular plate is essential, however, in order to justify the merits of the different designs. The stresses in a flat circular plate are tensile and compressive stresses along radial and tangential lines and direct shearing stresses. The shearing stress will not be considered for it will never be parallel to the main stresses caused by contraction. Considering the cover with radial ribs: When the metal cools it obviously solidifies between the ribs first, and the tendency will be to set up initial stresses between the ribs, Fig. 8(*a*). These stresses should be normal to the bisector of the angles formed by the ribs. Analyzing the stresses in the plate, it is found that the radial stress at the circumference is zero, but the radial and tangential stresses are equal and a maximum at the center. The tangential stress is not zero at the circumference, so it will be seen that

throughout the plate the tangential stress is larger than the radial stress. It seems obvious that the sum of the two forces, one initial and the other caused by the load, are combined algebraically and that failure occurs at a comparatively low external load.

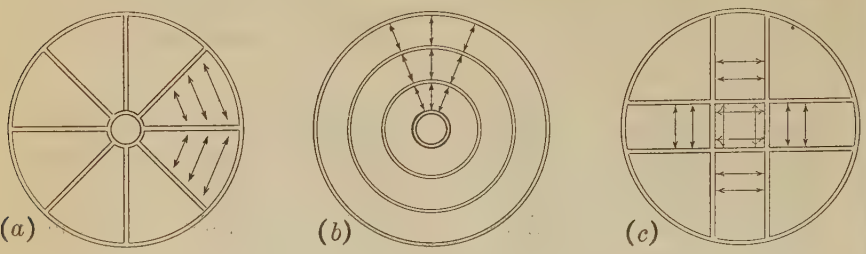

FIG. 8.

The cover with concentric ribs should have initial stresses on radial lines between the ribs, Fig. 8(*b*). This is apparently true, for covers made from the same heat withstood a 50% greater load than the cover with radial ribs. All test conditions were equal and the difference in weight was less than 5 per cent. This would bear out the fact that this combination of stresses is not to be neglected if the maximum strength is to be secured.

The contraction stresses should not be of such a serious nature in a cover with a rectangular system of ribs. This is particularly true if the ribs are deep, placing most of the flat cover in compression. These ribs should cause the stress to distribute in a manner similar to that in a square plate, Fig. 8(*c*). The contraction of the metal at the intersection of the ribs apparently causes relatively high initial stresses and is the one weak point in this type of design. The greatest strength, however, is obtained with this design.

TABLE 4.—SUMMARY OF TESTS.

Test number.	Record number.	Material.	Weight, in pounds.	Deflection, in inches.	Ultimate load, in pounds.
3	31 086	Cast iron	368	.....	32 400
10	"	" "	371	.....	30 170
2	"	"Semi-steel"	315	.....	27 630
6	"	" "	320	0.25	26 360
7	"	" "	329	0.25	26 360
1	"B"*	" "	338	.....	40 350
4	"	" "	330	.....	37 170
5	"	" "	335	0.22	43 530
14	"	" "	335	.....	40 350
15	"	" "	335	0.27	40 350
8	36 863	" "	419	0.14	68 980
9	"	" "	416	0.244	100 800
12	"	" "	419	0.118	53 400 (Not tested
17	"	" "	419	.....	96 000 to failure)
13	31 086	Cast steel	365	1.310	60 300
16	"	" "	...	0.540 (approx.)	60 000
11	25 370	" "	288	.....	66 750 (Yield point)
A	"	" "	288	.....	100 800

* Record "B", semi-steel cover with concentric circular ribs (Fig. 5).

SUMMARY OF TEST RESULTS

The results of the test indicate that the two designs for the new standards are satisfactory. The ultimate strengths of these covers are in excess of the

design load. For practical considerations the weight of the ventilating cover was increased from 288 lb. to 348 lb.; the distribution of the metal, however, would not increase the strength. The weight of the solid cover was reduced from 419 lb. to 340 lb., because a strength of practically 100 000 lb. is not necessary and the Company did not want a cover that weighed in excess of 350 lb. Table 4 shows the comparison of the ultimate strengths of the covers.

Conclusions

Table 5 indicates the relative strengths, in terms of cost, of a single design cast from different metals. Little advantage can be shown for one type of casting over another.

TABLE 5.—Comparison of Costs for Different Materials

Type of design.	Metal.	Ultimate strength / Cost
Old Standard	Cast iron	17.0
" "	"Semi-steel"	16.7
" "	Cast steel	18.3

Table 6 shows the relative merits of the different designs.

TABLE 6.—Comparison of Costs for Different Designs

Type of design.	Metal.	Ultimate strength / Cost
Old Standard (radial ribs)	"Semi-steel"	16.7
Revised design (concentric ribs).	" "	24.1
New Standard	" "	42.3

A most interesting fact brought out by the comparison in Table 6 is that a 25% increase in weight increased the strength 250 per cent.

The summary, Table 7, shows the saving in cost per installation obtained by this study.

The saving in cost for each installation ranges from 22.3 to 40.8 per cent. A considerable part of this was obtained by reducing the weight of the frames as they were exceedingly large, considering the actual load they transmit to the concrete of the manhole. City ordinances for paving require a certain height for the frame, otherwise the new frame would only be about one-half the height of the old frame. This would be possible as the new inner cover is 0.25-in. plate-dished to conform with the ribs of the cover.

The product "semi-steel" is ideal for the solid manhole covers because the cost is the same as for cast iron, although the tensile strength is 50% greater when approximately 25% of steel scrap is included in the melt. The new covers have been in service for more than two years and no failure has been reported.

TABLE 7.—Comparison of Total Costs

Covers.	Cost of Covers.	
	Old.	New.
0-Inch Solid Covers:		
Frames	690-lb. at $0.055 = $38.00	445-lb. at $0.055 = $24.50
Cover (outer)	320-lb at 0.055 = 17.60	340-lb. at 0.055 = 18.70
Cover (inner)	200-lb. at 0.055 = 11.00	84-lb. at 0.10 = 8.40
	$66.60	$51.60
36-Inch Ventilating Cover:		
Frame	690-lb. at $0.055 = $38.00	445-lb. at $0.055 = $24.50
Cover (outer)	630-lb. at 0.085 = 53.50	348-lb. at 0.085 = 29.60
	$91.50	$54.10
28-Inch Solid Cover:		
Frame	525-lb. at $0.055 = $28.85	360-lb. at $0.055 = $19.80
Cover (outer)	240-lb. at 0.055 = 13.20	226-lb. at 0.055 = 12.40
Cover (inner)	115-lb. at 0.055 = 6.35	54-lb. at 0.10 = 5.40
	$48.40	$37.60
28-Inch Ventilating Cover:		
Frame	525-lb. at $0.055 = $28.85	360 lb. at $0.055 = $19.80
Cover (outer)	390-lb. at 0.085 = 33.20	226-lb. at 0.085 = 19.20
	$62 05	$39.00

Acknowledgments

Report No. 402 by the Bureau of Tests and Inspection of the Pacific Gas and Electric Company contains all the data taken during the test. The Bureau was represented by Mr. A. D. Macintyre, who assisted in making the tests and collecting the data.

R. R. Cowles, Assistant Engineer in the Division of Electric and Steam Distribution, assisted in making arrangements for the test and collecting the necessary data in order to make this study on the covers.

G. H. Delarenelle, Assistant Superintendent of the United Iron Works, assisted in every possible way. The handling of the covers and the operation of the machine was performed by his employees. Equipment for the test was loaned by H. B. Hammill, Assoc. M. Am. Soc. C. E., and by the Pelton Water Wheel Company. The test laboratory of the University of California was used for making the tensile test to determine the properties of the metals.

DISCUSSION

M. M. O'Shaughnessy,* M. Am. Soc. C. E. (by letter).—The writer has been impressed with the attention and painstaking work that have been applied to these tests. Usually, in this class of work, stresses are first computed on a theoretical basis, and if these indicate thin sections, the designer at his own option arbitrarily makes the metal thick enough to allow for practical moulding in the foundry.

In the present instance, not only is the theory taken into account, but the manner in which covers in actual service stand up is observed, such covers being taken out of service and put in the testing machine. The loads for which they were designed, namely, 15 000 lb., plus an impact factor of one-third, seem to be ample, and closely agree with the present city practice of assuming 20-ton trucks with two-thirds of the load on the rear axle plus 50% impact.

The saving that has been accomplished is notable particularly the decrease in the weight of the frames. It would be of interest to have a plan of the old frame and of the new frame for the benefit of those interested in this subject, also the physical properties of the cast iron, semi-steel, and cast-steel. The paper does not state whether all frames and covers are machined in practice, or only for purposes of test.

Generally, considerable attention is paid to the economics of the larger structures in the field of engineering. Small equipment and appurtenances are frequently neglected; but, as shown in the final analysis in this paper, it would be well if more attention was paid to the smaller details entering into the daily operation and maintenance of engineering works.

T. J. Corwin, Jr.,† Assoc. M. Am. Soc. C. E. (by letter).—At the suggestion of Mr. O'Shaughnessy certain information is here given, which shows more clearly why changes were made in the designs.

The two frames are shown in Fig. 9. A comparison of the two will show that the new frame is of relatively simple design compared with the old one. By using a ¼-in. steel plate for its inner cover, dished to conform to the ribs of the outer cover, only one shelf was necessary to support both covers. The greatest saving, however, was made by reducing the area of the base, the side walls remaining approximately the same. This reduction of the base resulted from investigating the intensity of the bearing stress on the concrete supporting the frame. The stress is almost negligible with a uniform distribution of the load, and is approximately 300 lb. per sq. in. under the most adverse condition of loading for the new frame. The area of the new base is approximately one-half that of the old one. The new designs specify that the supporting edge of the cover and its seat in the frame shall be machined surfaces.

The physical properties of the metals used for the test are as given in Table 8.

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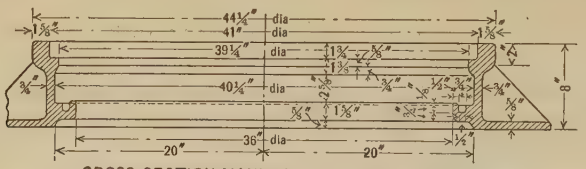
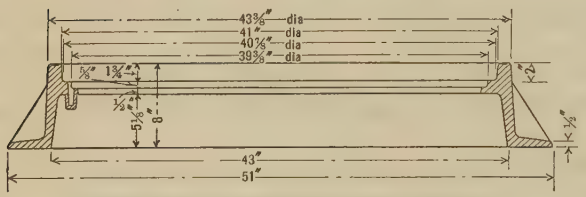
† Asst. Engr., Pacific Gas & Elec. Co., San Francisco, Calif.

No grey-iron castings were made for this test. The grey-iron covers were taken from service and no arbitration bars were available to check the



CROSS SECTION MANHOLE FRAME OLD TYPE



CROSS SECTION MANHOLE FRAME NEW TYPE

FIG. 9.

physical properties of these covers. It was assumed, however, that the melt conformed to the requirements of the American Society for Testing Materials for the medium grade of castings.

TABLE 8.—PHYSICAL PROPERTIES OF METALS USED FOR MANHOLE FRAMES AND COVERS.

Ultimate strength, in pounds per square inch.	Percentage of elongation.	Modulus of elasticity, in pounds per square inch.
CAST-STEEL TENSION TEST.		
67 800	25.	31 800 000
68 750	22.	35 000 000*
SEMI-STEEL TENSION TEST.		
30 640	0	16 000 000
30 000	0	15 200 000

Ultimate load, in pounds.	Deflection, in inches.
SEMI-STEEL CROSS-BENDING TEST.	
4 520	0.097
4 740	0.100
4 930	0.118
5 600	0.120

* Test specimen was cut from one of the covers after the cover had been tested to failure.

With regard to the economics of this investigation, the entire cost of the test was saved on the first order of the new type of covers and frames.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

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Paper No. 1653

ECONOMIC AND ENGINEERING PROBLEMS OF HIGHWAY LOCATION*

By W. W. Crosby,† M. Am. Soc. C. E.

With Discussion by Messrs. L. O. Marden, Charles B. Breed, Fred Lavis, George E. Goodwin, H. C. Boyden, John P. Hallihan, and W. W. Crosby.

"Economic"—like "efficient" and some other terms now in frequent use—seems to cover, more or less loosely perhaps, quite a range of meanings. Originally applying to "household" or "domestic" matters, and later pertaining to pecuniary means or concerns relating to income or expenditure, its use has grown—apparently quite properly with the development of the household or family into the community or Nation—to cover "the production, distribution and use of wealth (or resources)" and "to relate to means of living or arts by which human needs and comforts are supplied."

This interpretation is given to avoid at the outset any impression that the narrower sense of the word—such as "sparing" or "saving", not to say "mean"—might equally apply.

A few of these economic problems of highway location will occur to the engineer clearly, as, for example:

(1).—What "controls"—that is, what geographical points to be selected as termini or as intermediate points—shall be determined or accepted for the highway?

(2).—By whom shall they be determined?

(3).—In their determination, what weights shall be given to such factors as:

(*a*) The character of the highway needed—that is, the probable use to be met by it after completion—and the probabilities for permanence or development of that character.

* Presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926.

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(*b*) The sources of funds available for the construction of the highway.

(*c*) The relation of the location and usefulness of the highway to the other existing or probable means of transportation serving the same areas and foci.

(4).—What controls or locations shall be selected for purposes of:

(*a*) Avoidance of traffic congestion.

(*b*) Time-saving in highway travel.

(*c*) Decentralization of population and commercial activities.

(*d*) Property development.

(*e*) Convenience and pleasure in highway use.

(5).—Effects of existing laws and customs or lack of law and authority on the location.

(6).—Elasticity (or the lack of it) of local highway laws affecting revisions of locations.

(7).—Legal authority for the persistent protection of the public highway.

(8).—Control of the use of the highway.

(9).—Public education on this general subject.

Similarly, the engineering (technical) problems will include:

(10).—Comparisons of the results of direct or indirect alignment between controlling points, as to:

(*a*) Costs of construction.

(*b*) Costs of operation (maintenance and use).

(11).—The effects of grades on traffic costs, speeds, and operations, both for present and future motor traffic.

(12).—The effects of length on these same factors.

(13).—The determination of suitable widths to insure maximum satisfaction.

(14).—The regard to be paid to the provisions for automatic safety of traffic.

(15).—The regard to be shown for the speed (or time) factor for individual units, as well as for the average of the whole traffic.

(16).—Local physical conditions affecting construction or maintenance.

(17).—Modifications of normal construction details necessitated by the foregoing considerations.

These problems will be analyzed in the order given.

Problems (1) and (2).—Controls.—Modern road work in America has been confined mainly to the improvement of a small percentage of old roadways—roadways which for the most part had "just growed". The selection of the fraction to be improved has been made in most cases by political bodies, that is, by elected representatives of the voting population of the State, among whom the economist and the engineer have usually not been identifiable. In only one or two cases, to the writer's knowledge, has any perceptible influence

from an economist or an engineer been apparent in the selection of the State highway systems.

In other words, the economic problems of controls have been settled by politicians rather than solved by experts, and if these solutions have been really economic, it is merely a fortunate, and accidental, result. As many are beginning to realize, however, such fortuitous good results are not prevalent enough to warrant the longer neglect of this important matter. Some more scientific method must be reached, for determining between what termini main highway systems, costing millions of dollars, are to be built and through what intermediate points they may wander en route.

The railroads, as a whole, did better with their locations although some were fixed in considerable detail by political authority against the struggles of the less selfish advisers.

One reason for some of the extravagant or otherwise bad political locations is that no convincing arguments were presented against them. Usually the question of location came in the form of a choice from one or more routes between two points, the connection of which by a State highway was recognized as mandatory; and the strongest politicians won.

Until recently there has been little evidence that the economics of such a choice were considered or even presented. As a matter of fact even now there would probably be as many different styles of presentation as there were economic advisers, because of the newness and present uncertainties of the subject. As for the engineering side of the question, that, if it intruded, has usually been treated in so flattering a manner as to dismiss it completely. "The engineers will take care of that difficulty", has resulted in many a regrettable location.

In Maryland, the selection of the old routes was made under such a general authority and control as:

"Such a system of main market road connecting the various County Seats with each other and with Baltimore City as might reasonably be expected to be built with the funds provided". (The specific sum named was $5 000 000).*

The resulting State Highway System was a fairly logical selection from the existing establishment of old roads, although political influence warped the results somewhat and prevented the ultimate economy desirable. No authority was given the Highway Commission, however, to consider routes other than the existing public roads.

The propriety of the legislative action in this case was rather remarkable and seems to have offered an example to be emulated by other States, although it is not evident that many did follow that lead. To some extent the Federal Government, however, has pursued the idea.

Problem (3).—Factors Affecting Controls.—The main controls having been established, many problems as to the intermediate or minor controls of the locations arise, and with them Problem (3), involving factors governing such controls, appears:

(*a*) Obviously, if a highway for through traffic is being located, indirectness must be avoided even at the expense of by-passing an

* State of Maryland, Acts of 1908.

intermediate community. Equally, unless the highway is to be largely for industrial traffic, directness, at the expense of avoiding points of scenic and historic interest, or of failing to serve localities through which it runs, is not warranted.

(*b*) If the sources of the funds for the highway come solely from the State Treasury or a State bond issue, for instance, the general State interest in the location must take precedence over local or individual interest. If the cost of the highway is largely met by taxation on adjacent property, the reverse is true. If a highway is being financed as an industrial and commercial proposition, its location must be regarded in a different light from another the costs of which are to be met by a general tax on the sales of gasoline.

(*c*) Frequently the argument for locating a State highway to parallel closely a steam railroad is that such a location would reduce the cost because of the advantages in the supply of materials for construction. This is a pure fallacy. It would be much better to argue that State highways ought to run at right angles to railways. Of course, some conditions seem to warrant the supplementing of railway facilities by parallel highway facilities within limited areas; but considerable study and discussion are needed before principles can be well established in this matter, particularly in view of the unstable conditions now observable both as to present and future prospects in the matters of traffic developments, civic developments, and progress in mechanical equipment for movement of both persons and goods.

Problem (*4*).—*Social Aspects.*—

(*a*) Traffic congestion (even with the parking factor eliminated) is largely resolvable into a location problem, and would be better solved as such rather than as a police matter.

(*b*) The value of time in movements is becoming important and proper location is a fundamental for solving this problem. Time or speed is now a fourth dimension, added to those of length, breadth, and thickness (or grades) and applying to all solutions of highway location.

(*c*) Already decentralization of community life appears on the horizon, and its advent will be hastened by the daily developments in better means of communication and in facility of individual movement. Proper recognition of the facts demands that highway locations, over which much of this movement must take place, shall not oppose the other forces acting to increase the movement. The highways should really be located so as to co-operate with those forces.

(*d*) Contemporaneously, the opening of undeveloped property for use in connection with decentralizing movements as well as the development of this property demanded by the natural growth of population calls for suitable highway locations.

(*e*) As "Man does not live by bread alone", the convenience and pleasure of a people, naturally inclined to move about readily, furnished with facilities for movement, and urged on every hand to supply themselves with vehicles for individual movement at any and all times, will require suitable regard for highway locations to accommodate their needs. It is a mistake to regard highways as purely industrial means to a commercial end. The majority of highway traffic is now "pleasure traffic" according to the surveys of several States. It seems likely that it will not be otherwise for some time. One distinguished authority has written that the "saturation point" of motor vehicle ownership in America is to be calculated solely on the basis of time available for use of the car. As every one knows the curve of American leisure is upward.

Commercial highways, industrial roads, scenic highways, super-highways—all will present their own problems of location; but first—perhaps even more so than "the poor"—there will always be "with us" the proper location of the general purpose highway.

Thus far, in most cases of organized highway improvement, a proper conception of these problems has been prevented by too great a regard for existing roads and interference with, or lack of, a view of anything outside of them; and yet it will be seen that a proper solution of the economic problems of highway location, while calling for a careful regard for existing highways, requires vision beyond these and authority to be given into competent hands to produce, through proper channels, results in roadways outside existing public rights of way.

Problem (*5*).—*Revisions of Alignment.*—Unless State highway laws permit departures from existing rights of way and even the establishment of new ones, the hope of solving some of the economic problems of highway location is lacking. If road improvement by the State is to be within the limits of existing public rights of way, nothing but mere roadway improvement can be expected and that itself must be regrettably limited.

Problem (*6*).—*Limitation of Future Changes.*—Location solutions, forced to be settled to-day for an indefinite future, may often be different from those made for the present and the immediate apparent future, when the laws give some authority for making later changes that may then prove necessary. In other words, the amount of elasticity provided for successive action affects to some extent many location solutions.

Problem (*7*).—*Invasion of the Highway by Outside Interests.*—Similarly, protective action against common tendencies toward encroachment on the highway by private or special interests has often to be considered in determining details of the location. Unless warrant exists for the highway authorities to exercise specific protective measures, these may have to be attempted through suitable location alone.

Problem (*8*).—*Statutory Limitation of Use.*—Some problems of location also have to be regarded in the light of "use control" by the highway authorities. Lacking such control, a detail of location, such as width, which again

affects alignment, may require different treatment. Similarly, safety, use control, and location are intermingled. Automatic control and automatic safety are far superior to the artificially attempted variety. "Build the safety into the location" is a good slogan.

Problem (9).—Public Education.—Last in the list comes this economic problem of how best to proceed with the location to secure the necessary public education and to obtain results in the form of better roadways that will render a higher degree of public service and yield the most lasting quality of satisfaction. Such public education is feasible. Highway authorities, particularly highway engineers, are the ones to lead in it.

In 1911 the State of Pennsylvania showed some grasp of the economic problems of highway location by providing in the Acts of that year (establishing its State Highway System), the following:

"Whenever in the construction, reconstruction, maintenance, and repair of any of the State Highways it shall appear to the Commissioner that any part or portion of a State Highway, as now defined and described in this Act, is dangerous or inconvenient to the traveling public, in its present location, either by reason of grades, dangerous turns, or other local conditions; or that the expense to the Commonwealth in the construction, building, rebuilding, maintenance, and repair thereof would be too great or unreasonable, and could be materially reduced or lessened by a divergence from the road or route; the Commissioner is hereby empowered to divert the course or direction of same; and he may diverge from the line or route of same as herein described, in such direction or directions as in his discretion may seem best, in order to correct said danger or inconvenience or lessen the cost to the Commonwealth: Provided, That the said Commissioner shall first submit a plan of the proposed change to the Governor and that the same shall be approved by him."

Later, in 1921, a broader regard was evidenced by the following addition to the Highway Law:

"The State Highway Commissioner shall also have power, with the approval of the Governor, to establish the width and lines of any State Highway before or after the construction, reconstruction, or improvement of the same, not, however, exceeding the maximum width fixed by law for public roads. Whenever the State Highway Commissioner shall establish the width and lines of any such State Highway, he shall cause a description and plan thereof to be made, showing the center line of said highway and the established width thereof, and shall attach thereto his acknowledgement. Thereupon such description, plan, and acknowledgement shall be recorded in the office of the recorder of deeds of the proper county, in a separate book kept for such purpose, which shall be furnished to the recorder of deeds by the county commissioners at the expense of the county.

"No owner or occupier of lands, buildings, or improvements shall erect any building or make any improvements within the limits of any State Highway the width and lines of which have been established and recorded as provided in this section, and, if any such erection or improvement shall be made, no allowance shall be had therefor by the assessment of damages."

Under this authority the Highway Department has been operating as suggested in this paper. It has been felt obligatory to proceed conservatively and with careful regard of, and respect for, public opinion. Every effort has been made to have the public understand the reasons for disturbing established

conditions or for disappointing individuals in the provision of modern roadways on the State Highway System. It is believed some success has been achieved in enlightening citizens as to better highway location.

Problem (10).—Comparisons of Alignment.—Usually between two points of control there are several possibilities for the center line of the highway. A straight line between two points may be the shortest, but Moskowsky says it is "the most tedious distance". Traffic records show it is by no means the safest. It may not be the cheapest to build, nor even the cheapest to operate. The straight-line connection may not even be practicable. Still, its directness in most cases draws the final solution toward it "as the needle to the Pole".

Problem (11).—Effects of Grades.—Usually decisions as to alignment depend largely on the grades. As a matter of fact, the effects of grades on highway traffic of the present and of the near future is one of the most important primary engineering problems awaiting solution.

The older factors limiting highway grades, such as the ability and endurance of draft animals, the water-proof qualities of the surface, the absence of any speed factor, and even the slipperiness of hills, have wholly or largely disappeared on main roads. Nowadays there is plenty of power for grades of twice the former maxima, there are water-proof surfaces, fair control of speed, and far less danger from slipperiness. Only for the fact of uncertainty as to what the next twenty years may hold in the way of motor vehicle development it would seem entirely practicable to adopt a general rule to the effect that the grade maxima generally accepted as proper twenty years ago could be doubled. With the necessity for limiting widths and weights of vehicles, is not the motor train a development to be expected? And what will be its abilities on grades?

Passenger cars are now able to surmount grades of 12 or 15% if not too prolonged. Trucks, however, find difficulties with such grades (or even less) particularly in descending them. Are not improvements in braking (for instance) to be expected, which will relieve that difficulty? The revolution of railroad operations through the invention of a successful air-brake, is fairly recent.

Some excellent work toward determining the effects of grades on present-day traffic has been done by T. R. Agg, M. Am. Soc. C. E., among others. Professor Agg, however, admits frankly that his opinions are more tentative than conclusive. Of course, their relation to future traffic developments is problematical.

The importance of this matter of grades can hardly be over-estimated. On its proper determination depend solutions of alignment and many other technical problems of location.

Problem (12).—Effects of Length.—The effects of length as an economic problem have been noted; but they have their engineering aspects. It will be the province of the engineer at least to adjust the relative effects of lengths and grades in first costs and to see that the consideration includes the mechanical developments to be expected.

Problem (13).—Suitable Widths.—The protection of the public interests in the roadway will throw on the engineer the duty of establishing similar widths for the location of the highway. Even the alignment will depend more or less as to its details on the width established.

The fixing of the width of the right of way for modern highways is primarily based on the necessity as to the width for the roadway and its structures. Other factors also enter, such as the opportunity for footpaths (and possibly even for horse-drawn or slow moving vehicles) public service poles and structures, clear vision for fast traffic, reasonable provision for standing vehicles, even beautification, and convenient or recreational adjuncts.

It seems in many cases that the fixation of a proper width for the public right of way is necessary to the end of protecting the roadway itself. The "set-back" of building lines beyond the right-of-way limits on public highways outside municipalities is not, nor is it likely to become, a usual power in the hands of road authorities. Hence, the only way to accomplish the desired results seems to lie in the establishment of sufficient widths in the right of way itself.

Usually, the symmetrical development of the public right of way requires that the center lines of the roadway and of the right of way coincide. There are numerous exceptions, and often quite the contrary principle must prevail. Both these two latter considerations, therefore, will affect the alignment and the location.

Problem (14).—Safety of Traffic.—There is much discussion nowadays about safety of highway traffic. In addition to having safety "built into" the location as already suggested, it frequently is practicable to adjust engineering details of alignment, widths, and grades so as to provide a minimum of danger to traffic as a whole and thereby also largely abate the need for artificial or police control of the use of the highway. Engineering may even offset to some considerable extent the dangers from the reckless individual motorist. Wherever engineering may provide automatically a decrease of danger or an increase of safety to the traffic as a whole, that solution is naturally to be preferred to the one of personal regulation.

Problem (15).—The Time or Speed Factor.—The time or speed factor in location problems must be recognized by engineers. A curve entirely practicable for traffic averaging 5 miles per hour is entirely unsuitable for 25 miles per hour. Grades that increase time schedules between termini excessively without offsetting advantages of scenery or other compensations must be abandoned for other locations more agreeable and acceptable.

Problem (16).—Effect of Physical Conditions.—Local physical conditions affecting construction or maintenance must be considered by the engineer in his determination of the location. In laying out old roads engineers often ignored the effects of periods of high water, unsuitable subsoil, snowdrifts and ice, and similar circumstances, affecting cost and even usefulness. Proper regard for such local phenomena will often introduce a weighty factor into the location.

Problem (17).—Modification of Construction Details.—A broader and more forward-looking regard for better highway location by engineers quite likely will suggest some modifications of the present standards of construction itself that have grown up more or less fortuitously and unconsciously. These technical items will include the easing of curves (affecting the alignment) and the widening of the roadway at least on curves (both as regards the effect of the speed factor); the development of the width of the locations, to provide best for its utilization with regard to convenience and safety of traffic; and all other public service interest in the highway right of way, existing or likely to prevail. Some at least of these details will affect the location.

The opportunity offered for presenting this paper has seemed to the writer best improved by thus briefly enumerating some of the problems of highway location that have come up in his experience. Some of these many problems have not yet been satisfactorily solved except for a particular time and place. In other instances, the solutions suggested have been more expedient, perhaps, than scientific, although the effort has constantly been made to keep the science clearly in mind with a view to its best possible application.

Governor Pinchot of Pennsylvania in 1923 vetoed a bill making certain State highway locations by political selection, and Governor Smith of New York did so emphatically in 1924. Maryland, Georgia, and perhaps several other States, have avoided to some extent the political evils of selection of their State Highway Systems by legislative log-rolling. However, even where the location of the limited mileage of State highways was made by a non-partisan commission, that selection has been hampered and often rendered inferior by limiting—through lack of authority for doing otherwise—the selection to the existing old highways.

In other instances, taking Pennsylvania as illustrative of this group, the selections have been made by the Legislature itself. In Pennsylvania, although rather definite controls of the State highway routes selected were prescribed in detail, one saving clause of the State Highway Act of 1911, giving the State Highway Department power to relocate State highway routes relieved the situation extraordinarily. When to this clause were added the amendments of 1923 (as to widening of State highway rights of way) and of 1925 (legalizing the filing of a new right of way for protective purposes and future use), it will be seen that in some ways perhaps and despite any handicap of legislative establishment of controls, freedom of action in highway location in Pennsylvania has been greater than elsewhere.

In addition, the work of establishing the ultimate right of way is considered extremely important in Pennsylvania. Other States, it is to be noted, have envisaged it. Its execution, however—the securing of greater widths for the public right of way—is often an expensive matter. Always it is a difficult one. It must be approached carefully if disaster is to be avoided.

The ultimate width is inextricably involved with other details of location. Such a proposition as a legislative enactment that "all State highways shall be 100 ft. wide", even if legally sound, would probably prove enormously extravagant and eventually be unsatisfactory and disastrous unless coupled with sufficient authority to revise the alignments, that is, to relocate them, contemporaneously. Even then, with sufficient authority in proper hands for both relocation and widening, there should be some elasticity and recognition of differences in local circumstances and in needs for the widths provided, in order that the progress achieved, if slower and more gradual, may be more lasting and ultimately satisfactory both in the physical results on the highways and in the position of the highway engineer in the community.

DISCUSSION

L. O. MARDEN,* ASSOC. M. AM. SOC. C. E. (by letter).—Undoubtedly, as Mr. Crosby has stated, a great many old highways were located through politics. However, several trunk lines in New England were laid out by private companies called "Turnpike Companies". Those located during the last thirty years in many cases served the towns having the strongest political representation whether or not they were on the most direct route.

At present, one of the most heavily traveled roads in Massachusetts is the "Old Newburyport Turnpike". Plans have also been made by the Massachusetts Department of Public Works, Division of Highways, for a main trunk line along certain parts of the old turnpike road between Boston and Worcester. On the other hand many of these old turnpike roads have fallen into partial disuse for various reasons:

1.—The old central village of the township through which the road passed may have lessened in importance because of a new railroad village one or two miles from it.

2.—Many old mills located in remote locations, which were near the turnpike, moved to the railroad, or never rebuilt after the buildings were destroyed by fire.

3.—Old farms have been abandoned.

4.—The grades were too steep for modern travel.

The engineers who located many of these old turnpikes should be given due credit for their monumental work in a day when the country was young and the traffic demands were less stringent.

CHARLES B. BREED,† M. AM. SOC. C. E.—Mr. Crosby has not attempted to set forth any specific engineering or economic theories, but has confined his paper to a more general treatment of the subject. The speaker likewise, will avoid any mathematical analysis of the relative values of length, curvature, and grades in highways. Scientific analyses of these features of location may be ever so exact in logic and mathematics and yet, due to the scarcity or uncertainty of fundamental data, fall well short of giving results of great value.

The study of most engineering problems involves a knowledge of, and an application of, economic as well as engineering principles. It is often difficult and unwise to separate the two. The author has separated the purely economic from the purely engineering phases; but he has made clear that this separation is wholly artificial and for the purpose solely of emphasis and clarity.

Engineering facts can always be readily obtained by accurate measurements of the terrain; the possible alignment and grades submit to simple processes of surveying. The purpose of the location, however, is to furnish highway service of the character required on that particular road not only for the present, but for the future. It is not merely the design of a road

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† Prof. of Railway and Highway Eng., Mass. Inst. Tech.; Cons. Engr., Boston, Mass.

with flat grades and curves. The location is often spoken of as the one feature of the highway that remains for all time, or at least for several generations.

To predict the service required many years hence obviously requires superior judgment—it enters the realm of prophecy. To foretell judiciously the future service requirements of a highway obviously demands an intimate knowledge of its present requirements, the trend of development of the business and habits of the communities it will serve, the probable limitations of the vehicles used for transportation, and the effect of the engineering features, such as length, curvature, and grades on the cost of operating these transportation units. Were it possible to obtain these economic facts as accurately as it is to obtain the engineering facts the solution of the problem would be much more simple and definite. The principles of economics and of engineering are pretty well known; their application to a set of known facts is a process of accurate computation and thinking, but the facts themselves which form the premises are too frequently lamentably uncertain.

The history of railway location affords examples of this condition. Although there are many examples of poor railroad location in America the underlying fault was due in most cases to the lack of knowledge of the future sufficient to admit of a sagacious guess at the traffic to be expected. Railroads were constructed in many instances before any traffic actually existed; the service they rendered caused industry to create the traffic. As it grew in volume it became much greater than was anticipated by the most optimistic prophet. A new set of traffic facts and motive power facts then existed, which called for a radical revision in the location of parts of most of the larger railroads. Increase in traffic and development of the locomotive have made the old alignments economically unsound.

Thus it is that a poor location for one set of economic facts may be a good location for another; a poor location to-day may have been a good one 70 years ago. Engineering facts and axioms have not changed, economic principles have not changed, but the facts on which the economics was based, have been wholly altered. It is easy to criticize a location in the light of past facts, that is, judgment. Prophecy into the future cannot hope to reach the infallibility of judgment of the past.

In highway location the change in transportation methods from the horse to the motor vehicle has altered the whole economic program and has enormously increased the amount of traffic. It is difficult to predict far into the future with relation to the ultimate limits of the motor vehicle; people are living too close to the day when no one would have dared predict what their eyes are now seeing, for them to make sanguine guesses as to the future of highway traffic. Highways must be constructed, however, and the engineer must select the route.

The proposal that the economic and engineering problems in highway location should be under the control of the engineer, strikes a sympathetic chord in any association of engineers. A poor location is often attributed to the dominance of politicians of the past. It may have been largely due to the lack of vision of engineers as to the tremendous development in transportation

vehicles. Earlier settlers should not be criticized too promptly. It is not at all comfortable to feel that future generations may attribute present shortcomings in location to the dominance of the politicians of to-day, for that indicates that the engineer's influence in public affairs has been weak. Political influence does exist to-day and engineers have a very real responsibility in combatting the political selection of highway routes. That responsibility lies now, as it did then, in the apathy shown by most engineers in standing aside and permitting these influences to control.

Even where there is no doubt of its existence there are still great obstacles to successful opposition to it. The ammunition of the combat is publicity—it is a process of education of the public—the powers that control the selection of a highway route often also control the press. The knowledge of the facts and principles necessary to initiate and carry on the fight usually lies only in the mind of the engineer closely associated with the project. One naturally asks the question—would his position thereafter be a secure and happy one? It is a comforting thought for the engineer who courageously combats improper influence that public confidence in his superior ability to solve such problems is increasing because that confidence has been justified.

Errors in new location and in the improper selection of existing routes for improvement are a waste of public funds and should be avoided by removing if possible their cause, which usually is undue influence. When all improper or selfish interest is removed from effecting public improvements, presumably the millenium will have arrived. It is obviously the engineer's duty to assist in preparing the world for this important event.

Modification of the location of present highways for the most part consists of flattening curves and grades, but preserving much of the old location both in line and grades. This clinging to the old location and width of right of way may be carried too far through fear of apparently giving undue weight to a prophecy into the future. The public improvements that have been courageously developed along broad lines have seldom been regretted. It is the makeshift and niggardly policies in public affairs that have been expensive. Progressive cities have resorted to radical cures of highway transportation problems by city surgery. Why should not progressive States adopt the same policies?

Because of the ignorance of economic facts that may exist in the future it is advisable to select a location to-day which is amply wide and which with respect to grades and curves will admit of the necessary improvement to meet almost any set of conditions that its use or service may later demand. All these improvements need not be made at once; some that are possible may never be made. That is what American railroads did; they have recently been revising grades and adopting flatter curves. If the highway location is made wide enough at curves they can be flattened when required. If the summit is located in earth rather than in ledge, its grade can be reduced by excavation and filling when the traffic justifies the expenditure. Both low and high gradient roads are the truly economical roads under certain conditions. Such a policy as this produces highways of the greatest good to the greatest number.

Any feature of location that will affect the ultimate volume of traffic should be considered, because the purpose of superior location is to stimulate traffic—to give the maximum mobility to highway transportation.

Decentralization of community life appears to be a permanent rather than a transitory movement in America. If this premise be true, then it should be given consideration in highway location. Certain it is that America is mobile. Evidence of this lies in the fact that it has one passenger automobile to every seven persons. It is also evidenced by the recent and rapid development of wayside camping grounds and cottages for overnight accommodations. Had not roads been improved, community decentralization would not have progressed.

The location of a commercial or industrial road should be safe and direct, and devoid of long, steep grades. Its selection is a problem of economic compromise or adjustment. To keep the grade below a certain maximum often involves additional length and curvature or else excessive grading costs. In few instances can the ideal be reached; and if reached to-day, who can be certain that it will apply to to-morrow's traffic?

Through routes are often diverted from the direct routes for the purpose of making use of existing roads approaching and traversing prominent intervening cities and towns. As a result of this practice, through traffic is slowed down in passing through the intermediate cities, and the more congested parts of the municipalities on this route are subjected to the hazards of this through traffic. To make such through routes more direct by by-passing the intermediate towns, and to build short, well-paved feeders from the through route to the towns for commercial purposes, would seem to be in many cases the proper way to produce the maximum highway service. Such a policy, however, is frequently met with opposition from the retail storekeepers of the towns, who desire the maximum of traffic to pass their doors. It is not the motor-truck traffic that concerns them, but the fact that on all these direct commercial routes there is a great deal of passenger and tourist travel—a fact that the locating engineer should not fail to consider.

Deviation of the main road so as to pass through intermediate towns is usually more costly to transportation, all factors considered, than the use of short branch roads. Which course to pursue depends on the relative volume of through and way traffic, for the road should best serve and stimulate traffic.

Serviceable and attractive roads cannot be obtained unless they are planned. In the esthetic phase of road betterment, the endeavor should be to conserve Nature's beauties for the convenience and enjoyment of man; it should not endeavor to create them.

Roads suitable for commercial use are generally also suitable for passenger vehicles. There is nothing criminal in making a commercial road beautiful. The roads of heavy motor-truck traffic frequently have heavy passenger traffic. This is certainly the case in New England.

The present tendency is directly toward placing the cost of highway construction as well as maintenance on the highway user. In line with this logical tax, it becomes more necessary and reasonable that the user shall be pro-

vided with safe roads that will give the maximum service per dollar of expenditure. In so far as possible, automatic safety should be put into the location. This point cannot be too forcefully emphasized.

Many States of large area and limited funds are obliged to spread expenditures to such an extent that logical development in permanent locations and with permanent safety can only be accomplished by avoiding in the selection of the yearly program those proposed improvements that may militate against a courageous and logical solution of a location or safety problem at some future time. When a State is forced to that policy, there can be little hope for rapid progress toward permanent betterment.

The location of a scenic road is often simple. Provided the road is safe, its grades may be steep and its alignment crooked. If it is a choice between a scenic route on a new right of way and an ugly road over any of the existing routes, the only way to obtain the service desired is to adopt the new location. To spend public funds to build a low-grade direct route instead of a safe winding and scenic road for pleasure driving would be the height of folly.

The approach to that phase of highway location not affected by political influence or esthetics (in this paper called the engineering phase) logically follows the method adopted by the late A. M. Wellington, M. Am. Soc. C. E.* Such a treatment involves the study of the effect of length, curvature, grades, and rise and fall on operating costs. Mr. Wellington properly separated these subjects into those which affect the number of trains required for a given traffic and those which affect only the cost of operating a given train.

The maximum grade limits the total load that can be hauled by a given locomotive. For a given traffic, therefore, it fixes the number of trains required to move it. There is nothing gained by designing locomotives with the shift-gear feature for using a low gear when climbing a grade, because the tractive effort of any locomotive at low speeds is limited by the adhesion between the wheel and the rails. Generally speaking, locomotives are designed so that at low speeds there is sufficient power to slip the driving wheels.

On maximum highway grades the coefficient of friction between the wheel and the road surface is about three times that on railroad tracks. The result is that slipping of the wheels of a motor car on a firm, dry road is not at all likely on grades that would be considered reasonable on any State Highway. The shifting from a higher to a lower gear transmits the motive power to the wheels of the motor vehicle at a slower rate, with the result that the climbing of steep highway grades is attended only with loss of time and increase in fuel consumption.

The outstanding difference between the economics of railway and of highway location lies in the fact that the low grades on a railroad are of paramount importance, whereas there is no feature in highway location—length, maximum grade, curvature, rise and fall, or width—that is so critically important to a highway as the maximum grade is to a railroad.

All the effects of these various elements in location, in operating costs on highways, fall into the second class, namely, those which affect the cost of

* "Economic Theory of Railway Location."

operating the unit of transportation. In none of them is there any danger of actually stalling the vehicle or of making it physically imperative to use more vehicles.

In comparing two possible locations—one the direct and logical highway alignment from the standpoint of good economics and good engineering, the other the more popularly favored route, even if the effect of the latter on operating costs may be considerable—the degree of prominence of these unfavorable elements of greater length, steeper grades, and sharper curves, in comparison with the effect of manufactured public opinion, public apathy, personal interest, and the like, is relatively much less than in the case of a railroad. There is a grave danger, therefore, inherent in the relative strength of the forces tending to fix a highway location that undue weight of half formed or improperly formed public opinion may outweigh in the minds of legislators and public officials the sounder engineering and economical principles.

From all this one might be led to the erroneous conclusion that the only elements of highway transportation cost affected by the four different elements of location are the variable costs, such as fuel, lubrication, tires, repairs, and time—that the fixed charges, such as taxes, registration fees, interest on money invested, and depreciation, are not affected. Any element of location that increases the time consumed decreases the miles per daily use of the vehicle and therefore theoretically, and in some cases actually, reduces the number of miles per year for that vehicle. Obviously, this increases the total cost per mile for those charges which fall in the class called fixed charges. A truck which operates only 15 000 miles a year will pay exactly as much tax, registration fee, insurance, garage rent, and interest on investment as one traveling 20 000 miles.

These enormous expenditures and their tremendous effect on industry, commerce, and finance, require thorough investigating by scientific methods—more appropriately by the Federal Government—to measure the cost of length, curvature, and grades, and of rise and fall, by outdoor laboratory tests over selected routes with calibrated vehicles of like design and condition and operated by drivers of like habits.

First, establish the fundamental facts before disseminating them. Only the fundamental principles are known at present. To-day the best engineering judgment is directed toward this problem. Searching for the facts will certainly improve the judgment. The result should be a location selected as intelligently as the type of pavement.

The U. S. Bureau of Public Roads has investigated the distribution and destructive effects of traffic loads on various pavements and subsoils; it has investigated certain structures and the properties of materials of construction. In addition, among others the Bates Road Test and the Pittsburg Road Tests were extensive and expensive studies of the behavior of pavements under intensified traffic. Further, The National Research Council is co-ordinating facts from experiments. Probably no such extensive study of the cost of different elements of location has yet been systematically made as in the investigations of pavement design, construction, and maintenance.

FRED LAVIS,* M. AM. SOC. C. E.—The application of any economic theory of highway location is even more difficult and complicated than is the case in railway location, on which latter the former must necessarily be based.

The problem of handling the traffic that passes back and forth, to and from New York City and the South and West, through the Metropolitan Area of Northern New Jersey, has long been recognized as one of major importance. There is not only the local traffic between New York City and the manufacturing and residential districts of the Area, but also the through traffic to and from the West via the State of Pennsylvania, the traffic to and from Philadelphia, Pa., Baltimore, Md., Washington, D. C., and the South, and the traffic to and from the New Jersey coast resorts.

The rate of increase in this traffic is shown by the following traffic counts made at Rahway, N. J., on the Lincoln Highway. They show a 24-hour count on a day in July for each year:

1921	12 153	1924	26 573
1922	16 610	1925	No count
1923	18 314	1926	43 735

Other counts show similar increases on other routes.

Up to the present (1926) the traffic across the Hudson River has been handled on the various ferries and more or less distributed, but when the Holland Vehicular Tunnels under the river are completed and in operation a large part of this traffic will be concentrated at one point in Jersey City, N. J. The congestion in Newark, N. J., is already so bad that not only is the through traffic delayed, but local business is greatly hampered.

To meet this situation the New Jersey State Highway Commission has planned and is building an entirely new highway from the New Jersey end of the Holland Tunnels through Jersey City and Newark to the westerly side of Elizabeth, N. J. The length of this highway will be about 13 miles and its cost will exceed $30 000 000.

It will have a roadway 50 ft. in width; there will be no curves of less than 1 000-ft. radius, and the gradients will not exceed 3.5 per cent. There will be no grade crossings with other highways and, of course, none with railways. Connection will be made at suitable intervals with the most important streets and highways which this new route crosses, by means of ramps, entering and leaving in the direction of the traffic.

The 50-ft. width of paved roadway will provide for five lanes of traffic, two in each direction, with one spare for emergencies, and it is expected that the traffic will be of a volume approaching 20 000 000 vehicles annually.

It will be realized, therefore, that the question of location and design is one of prime importance in a project of this nature, having more of the characteristics of a four-track main trunk railway through a densely built-up region than of an ordinary street or highway.

The route, of course, must be laid out primarily to serve the traffic, to take the people who desire to use it as closely as may be to and from the places between which they desire to travel. It must be laid out with a due regard

* Cons. Engr., Jersey City, N. J.

for the cost of acquiring the necessary right of way and to avoid unnecessary length, rise and fall, and curvature. The costs of construction and of right of way must be balanced against the increase of operating costs due to increase of length, rise and fall, and curvature. In designing this route it was found that data in regard to these latter items were very scarce, but studies were made and formulas developed to determine the differences in operating costs.

As in railway location, any theories of this kind must be developed and used with great care and judgment—mere mechanical mathematics are worse than nothing; but, as in all branches of engineering, the mathematical formulas should be developed and used as an aid to good judgment.

GEORGE E. GOODWIN,* M. AM. SOC. C. E. (by letter).—To one who has given much time, and even more thought, to highway location, this paper appeals as presenting, in a broad way, the principal highway location problems. Necessarily, not all highway projects are affected by many of the problems listed; also, some projects might have certain important affecting conditions not mentioned in the paper. This would be especially true of scenic mountain highways in the Western United States.

The principal difficulties in the way of correct highway location lie in the present methods of considering and determining the project in question solely in relation to itself and the immediate effect that it will have on the communities served. Frequently immediate benefits would be as great and ultimate benefits much larger, and more lasting and far reaching, if the project was studied and outlined in relationship to the other roads of that section and of the State system, and even in some cases of the National system, of highways; not only to the highways that are now improved or about to be improved, but also to the logical routes for important through highways that are reasonably sure to be built in the not distant future. Such considerations of a project can prevail only when the matter has been removed from the realm of politics and local community greed and jealousy.

Obviously, it is impossible for elective or appointive laymen to consider fully and decide wisely all the various economic and engineering problems of highway location and improvement, and as long as the engineering expert's studies and activities are directed and his findings approved or disapproved by a commission of political appointees with the ideas of whom the engineer must be in harmony, the results are bound to be mediocre. There are, of course, some wonderful exceptions to these conditions. Some State executives and commissioners have been broad and visioned men, considering the highway system from the viewpoint of the State at large, and have insisted that the highway systems improved be economical for the whole State in both engineering and commercial aspects. As a consequence in some States the highway systems have been developed along economical and logical lines. Public education on this subject is bound to bring beneficial results.

While all the physical conditions and standards as to length, alignment, etc., presented in the paper are important, the writer agrees that altogether too much stress has been placed in the past on limitation of gradient. What

* Cons. Engr., Hood River, Ore.

might be practical and necessary as a standard of limitation for alignment or gradient under certain conditions of topography, traffic, etc., would be absolutely wasteful and unnecessary under other widely dissimilar conditions. In many States more miles of equally useful, and oftentimes more pleasing, highways can be had if greater latitude is allowed in these matters both by the State Highway Departments and by the Federal Government. Statistics of motor accidents show that more accidents have occurred on relatively straight and level reaches of roads than on those having relatively steep grades and sharp curvature. Although, of course, neither steep grades nor sharp curves are desirable and should be avoided wherever practicable, it is a waste of public money and poor engineering to insist that all classes of roads in the system of a mountainous State should have, say, a maximum gradient of 5% and a minimum radius of 300 ft. for curves. Likewise, under certain conditions, standard widths of 24 ft., or more, are impractical.

As Mr. Crosby has brought out, all factors must be fully considered for an economic solution of these problems. In highway location, as in any other improvement work, true engineering economy consists in securing that which is best and obtainable for the funds, for the time being, and for the future as it may be visioned; and, while in some cases this result may be attained, it is probable that even the greatest successes of the present will afford ample reason for criticism by the engineer of fifty years hence, who will see only the results in the light of that time, but be unacquainted with the limitations and requirements that guided the original builder.

H. C. Boyden,* M. Am. Soc. C. E.—Referring to the real solution of many of the problems of highway traffic, the statement has been made that "to enter into an economic study of the location of highways, we have to enter into the realm of prophesy".

The engineer who is the best prophet is also the greatest dreamer and one cannot drive over American highways without realizing that they have been laid out by realists and not by dreamers or prophets. Most highways have been located with regard to the conditions existing at the time, without any serious attempt to provide for future traffic.

Who can prophesy what the volume of traffic will be fifteen years from now, any more than the traffic of to-day fifteen years ago could have been prophesied?

Only a very few years ago the (so-called) saturation point for automobiles was set at 17 000 000, and there are more than 20 000 000 already (not quite 1 per family) with no signs of the end. Now the automobile manufacturers, who are dreamers as well as hard-headed business men, are planning their future and spending their money on a basis of 2 cars per family. Are the engineers of to-day dreaming the same dreams and building accordingly? Quite evidently not.

A college professor, in discussing this subject, stated that, "before this dream comes true we will be traveling and carrying freight by airplanes". Undoubtedly, this dream will also come true, but history shows that a new

* Lecturer, Celite Products Co., Los Angeles, Calif.

form of transportation never lessens the use of any economical means of transportation used previously. The building of elevated railroads did not lessen the travel on the surface lines, nor did the building of subways put the elevated and surface lines out of business, as was prophesied.

Fifteen years ago, when California's system of State highways was started, no one dared dream that the car mileage on the roads of the State would be nearly 15 000 000 000 in 1927. Who dared dream that the time would come when the people of Illinois could spend $300 000 000 for a paved highway system of 9 000 miles, could pay the interest charges, the sinking fund, and the maintenance costs, and, at the same time, effect an annual saving of more than $80 000 000?

In locating highways two factors should be used as guides, namely, speed and safety. The end of "maximum speed" laws is in sight, because such laws are both economically and psychologically incorrect. Present and future highways must be made to carry more traffic than they are carrying to-day, and this can never be done with the existing speed limits.

The killing and maiming of the hundreds of thousands of Americans annually by automobiles is not because of excessive speed in driving, but because the highways are seldom built to carry the volume of traffic they must carry to-day. How much less are they built to carry the traffic of one's dreams, when these dreams come true?

John P. Hallihan,* M. Am. Soc. C. E. (by letter).—The author has brought out very forcefully the economic and engineering problems in highway locations, which have been ignored to a considerable extent in many of the roads that have been built in the last ten years. It is only within that time that the question of volume of traffic on main highways has become of very great importance, except in the immediate environs of large cities. Now, with the whole nation "on wheels," and with the automobile habit increasing by leaps and bounds, the question of volume must be given due consideration, and the "ox-cart" and "man-and-horse" yardsticks formerly used for the width of roads must be finally discarded.

The cost of a 20-ft. pavement is about $30 000 per mile on the average, and, generally speaking, in the National highway system, such a pavement is laid on a right of way of 66 ft. If the road were always to remain a country road of light traffic the width of the right of way would perhaps not have an important bearing on the location, but the moment that a settlement of any kind grows up along this road the width of the right of way becomes insufficient, and this insufficiency becomes more marked as the settlement progresses from a village to a town and from a town to a city.

The writer is of the opinion that no main transportation artery in a city, or between two cities of importance, should have a right of way of less than 120 ft. Such a width, after deducting one-quarter of the right of way for sidewalk use, would leave 90 ft. for pavement, giving room for four lanes of travel and a parking lane in each direction.

* Chf. Engr., Rapid Transit Comm., Detroit, Mich.

It has been demonstrated that the 120-ft. right of way carries certain advantages. The traffic capacity per lane increases rapidly with the width of street up to four lanes, but with greater widths at a much slower rate. If, in a congested district in a large city, it is desired to separate the traffic on any thoroughfare from that of a cross-street, a width of right of way of 120 ft. permits the two center lanes, one in each direction, to be elevated or depressed, without material damage to the abutting property as would be the case if the grade separation were attempted in a street of lesser width.

In a large city demanding rapid transit facilities, a width of 120 ft. between building lines for main thoroughfares increases greatly in economic value. It permits the construction of four tracks underground in the most economical manner because of reduction of underpinning costs, and, at the same time, permits more convenient arrangement of loading platforms at stations.

Considering the fact that over any considerable mileage of road in a fairly well-settled region the average cost of right of way will not exceed, say, $500 per acre, the addition in each mile of 6.54 acres required to obtain a 120-ft. right of way instead of a 66-ft. width would amount to only $3 270. This amount would add about 11% to the capital investment of $30 000 in the 20-ft. pavement. Under such circumstances no economist would question the wisdom of protecting the future usefulness of the investment by obtaining the additional right of way before laying the first pavement, nor of making that provision for the future a part of the cost of the original construction.

Mr. Crosby also draws attention to the fact that the volume of traffic to be carried warrants fitting highways more closely to the topography. Certainly in view of the probable increase in ownership of automobiles in the United States to one for every three persons, within the life of any road now built, the expenditure of considerable sums to eliminate sharp curves and steep grades on important highways would seem to be justified, as well as the departure in location of the most important arteries from the strict adherence to the section lines, as is now generally the case. On main highway routes it would be well to consider the desirability of by-passing the cities along the route at a convenient distance, near enough that they would be readily accessible, but far enough away that the through traffic would not be compelled to set up interference with the traffic of the city itself.

The warning sounded by Mr. Crosby, that the location of any highway of the present should be considered more carefully with relation to the future volume of traffic and to the lessening of interference on main highways with traffic of subordinate roads and of cross-roads, is very timely, particularly with respect to the highway development in the environs of large and growing cities.

W. W. Crosby,* M. Am. Soc. C. E. (by letter).—The writer, in this paper, attempted to cover a broad field. Similarly, the discussions have exhibited diverse, yet important, points of view.

If the treatment in the paper seems "general", as stated by Professor Breed, it may be that the almost insurmountable difficulties of a more specific dissertation were apparent. As he points out, "scientific analyses of these fea-

* Cons. Engr., Baltimore, Md.

tures of location [length, curvature, and grades] may be ever so exact in logic and mathematics and yet, due to the scarcity or uncertainty of fundamental data, fall well short of giving results of great value" now; but he probably will agree with the writer that persistent effort should be made toward supplying the lack and toward clearing up the uncertainties, and that one good method of attack is a frank analysis of the situation.

A major object of the paper was to bring to the attention of a most competent body of men the complexity and importance of certain problems that will count so heavily in the future of the Nation and of the profession. The writer has studied these momentous problems for some time, but frequently cannot—like others—"see the forest for the trees in the way". Therefore, an "airplane view" may offer some help. Too often for the sake of immediate results, now admittedly unsatisfactory, it has been the practice to cut laboriously through each tree. However, if the objectives to be reached are located first by surveying the whole field, may not the individual obstacles then be overcome with economy and more lasting satisfaction?

The broad view, necessary for actual results, is a responsibility of the profession, while it is necessary for all engineers to develop, by attention, thought, and individual effort, the facts, reasoning, and methods or conclusions, that are needed for the solutions. Some engineers may have to "dream" and even to become "prophets".

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Paper No. 1654

NORTH CAROLINA BITUMINOUS EARTH ROADS*

By William B. Catchings,† Assoc. M. Am. Soc. C. E.

With Discussion by Carlton N. Conner, Assoc. M. Am. Soc. C. E.

Soil roads in North Carolina include sand clay, top-soil, shale, chert, marl, spar, etc. Bituminous treatment of these roads gives results in appearance closely resembling sheet or sand asphalt, with practically no failures. During 1925 approximately 400 miles were completed.

Before attempting to treat a road, it is essential that the sub-grade and surface be put in condition to withstand the traffic it is expected to carry. The finished surface will ride no smoother nor will it carry any heavier vehicles than the base provides; therefore, every precaution should be taken in strengthening the base and all possible care used in putting the surface in as smooth a condition as possible. The entire road should be gone over and all weak places repaired with good soil or gravel. Skin-patching alone is not sufficient, as in nine cases out of ten the patch does not bond with the base and, later, scales under traffic, causing a failure in the surface. All curves of 900-ft. radius or less should be banked.

After the base has been patched, reinforced, and built up, and the curves have been banked, the entire surface should be scarified, machined, and kept dragged, retaining a crown on tangents of from $\frac{1}{4}$ to $\frac{1}{2}$ in. per ft., until the base is thoroughly compacted under traffic. Longitudinal dragging from one side to the other rather than from both sides to the center is recommended. A sectional drag that will follow the contour of the surface is better than one that cuts the quarters only. This preparation is best accomplished in the spring.

During the winter, sand for surfacing is kept in stock piles, $\frac{1}{2}$ mile to 1 mile apart, to avoid delays during the construction season. Before applying it

* Presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926.
† Constr. Engr., State Highway Comm., Raleigh, N. C.

this sand is hauled out and piled along the side of the road, and "cut back" in a windrow. This allows it to dry quickly and places it in a convenient place for spreading.

Prior to the application of the first coat of bitumen, all loose material is removed from the road surface, first by a road machine and then by a rotary street sweeper. A specially prepared tar is then applied at the rate of 0.3 or 0.4 gal. per sq. yd. and covered with sand to the extent of 12 to 15 lb. per sq. yd. After about six hours this surface is dragged with a sectional broom made of steel bristle push-brooms. This distributes the sand uniformly giving a smooth surface. The prime or first coat is allowed to set under traffic for from 5 to 10 days, in order to allow the volatile oils to evaporate.

All loose sand is then swept to the side and 0.4 to 0.5 gal. of a specially prepared asphalt is applied and covered with 30 to 35 lb. of sand per sq. yd. After 4 or 5 hours this coat is also dragged with the broom, which continues with additions of more sand until the asphalt hardens and will hold no more sand.

During 1924 and 1925 asphaltic oils were tried ranging from 55 to 65% asphalt (100 penetration); some straight penetration asphalts and several tars were also used with varying results. From these tests the present materials and methods were determined.

DISCUSSION

CARLTON N. CONNER,* ASSOC. M. AM. SOC. C. E. (by letter).—The author all too briefly describes one of the rapidly developing and improved methods of increasing the traffic capacity of a low type road at a moderate cost.

There are in the United States more than 368 000 miles of surfaced rural highways which in their present condition are suitable for only about 500 or 700 vehicles per day. This is nearly 80% of the total surfaced mileage; the remaining 20% are surface treated or surfaced with many types of treatments and paving up to and including Portland cement concrete pavements. The total mileage of the country is estimated at more than 3 000 000, of which approximately 85% has no surfacing; 10% has a low type of surfacing, such as the sand clay, top soil, etc., mentioned by Mr. Catchings; 4% has an intermediate type of surfacing ranging from surface treatments up to and including bituminous concrete on stone base; and less than 2% has a high type of surfacing, such as Portland cement concrete base with bituminous, brick, or block top course, and plain and reinforced concrete pavements. The bituminous earth roads of North Carolina are a step in progressive or stage construction which changes the low types of surfacing, consisting of sand clay, top soil, etc., to the intermediate type.

A highway surface may be considered as low type, intermediate type, or high type, depending primarily on its traffic capacity, and secondarily on its first cost, maintenance cost, and salvage value. The type used in North Carolina has been classed in other States as a skin surface treatment and would fall into the class designated as a medium or intermediate type.

There are of course several kinds of surfacing which may be properly classed as intermediate types, such as the mixed-in-place surfaces, sand asphalt, veneer macadams, and others which to a large extent utilize local materials for their base course and local or shipped-in materials for the wearing or top course. A majority of the intermediate type surfaces have bitumens as binders, tar is frequently used as a prime coat, and asphalt for the second and subsequent treatments. In some States tars are used almost entirely, whereas others are inclined to use asphalts. Freight rates and deliveries are sometimes deciding factors in the selection of the class of bitumen.

In New England this type of surface treatment extends over about 2 000 miles of road, using the local gravel as aggregate and tar as a binder. In Long Island and New Jersey are found hundreds of miles of road having the local sand loam mixed with asphaltic oils. In South Carolina, Virginia, Tennessee, and Pennsylvania, as well as in some of the Middle and far Western States, similar medium-type surfacings are being investigated, used, and developed. The actual materials vary with local conditions, but the objects are similar in nearly all cases—to improve the low type surfaces by the addition of a low cost wearing surface.

* Chairman, Investigation of Low Cost of Improved Roads, Highway Research Board, National Research Council, Washington, D. C.

These new surfaces, such as those developed in North Carolina, will increase the traffic capacities from 500 or 700 vehicles per day to 1 000 or 2 000. There are instances, such as in Long Island, where a daily traffic capacity of more than 5 000 cars and light trucks is claimed. It is not to be presumed that these surfaces will ever take the place of, or even compete in traffic capacity with, the high types of paving, but they will undoubtedly fill in many a gap and missing link of highway between objectives on trunk lines, as well as form an important part in the construction of feeder roads, of development roads, and of roads that carry a large seasonal tourist traffic.

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Paper No. 1655

RECENT DEVELOPMENTS IN CONCRETE PAVEMENTS*

By H. Eltinge Breed,† M. Am. Soc. C. E.

With Discussion by Messrs. A. T. Goldbeck, Elmer G. Hooper, T. M. Ripley, W. C. Hammatt, R. W. Crum, Carlton N. Conner, H. C. Boyden, C. Raymond Hulsart, and Edward Godfrey.

The building of concrete roads seems to have entered upon a stage of development comparable to organic evolution; the whole is being modified to an extent that cannot now be foreseen, by changes here and there in detail, often apparently unimportant, often tentative. The main advance is an increasing definiteness about the limits of scientific knowledge. Engineers perceive more clearly than they used to, the hither boundary of what they do not know, and are scientists enough to try to keep pushing that boundary farther off, by investigation, research, experiment, even by trial and error in practice. In other words, while all the factors entering into a problem of highway design are variable, the tendency is toward a mathematical, rather than an empirical, solution of it by trying to fix the limits between which the variables may work.

For instance, by practical tests and the application of the theory of design, there has been evolved a safe formula for the design of corners. Through laboratory research the essentials for obtaining more uniform concrete, such as quality of ingredients and methods of mix, have been brought within greater control. Through soil analysis and research, engineers are learning more about the supporting power of soils. Definite facts have also been obtained through tests, research, and observation, of which the following are examples: "Crack reduction is more economically accomplished by the use of steel reinforcement than by additional thickness of concrete"; and "a greater

* Presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926.
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reduction was afforded by small steel members closely spaced than by larger members widely spaced".*

Such are a few of the results available from undertakings of recent years. How much their influence on present practice will form the basis for further research is one of the guesses that add zest to life—and engineering.

Confusion may be avoided by discriminating sharply between the design of a rural road and that of a city street. The most important difference is in the bearing power of the subsoil on which the pavement rests.

Before the pavement for a rural highway can be designed, there must be a drainage system to take care of surface water by diversion ditches, slopes, side ditches, and culverts; and to take care of sub-surface water by cut-off drains and outlets. Such drainage must be amplified where necessary by the building of a suitable foundation with proper outlets, by the water-proofing of shoulders, and by the providing of drains along the pavement edges. It is most important to prevent the surface water that runs sideways in both directions from the road from concentrating along the edges where it weakens the supporting power of the sub-grade material or adds to its expansive force and to the consequent stress in the pavement slab.

On the other hand, the engineer who designs a city or village street usually has an umbrella over his whole subsoil surface composed of roofs, walks, curbs, gutters, and the pavement itself, to keep the surface water away from the sub-grade. Sewer, water-gas, and other corporation trenches also act as cut-offs for sub-surface water. Furthermore, the pavement usually has a sub-grade compacted by years of usage, often consisting of material with marked stability. Some of the developments to be mentioned are equally applicable to rural and urban works; a few are more important for rural work; and all need to be mixed with shrewd judgment of foundation conditions.

Cracks in concrete pavements have been so costly that many methods have been tried to eliminate or decrease them. Through the use of longitudinal division of the slab, of steel reinforcement, and of limited slab length, the desired result has been accomplished, provided, of course, that the other factors essential to the construction of a good pavement are of such design and under such control as to give a uniform result.

Evidence shows that a slab length between 40 and 60 ft. for rural work, with from 30 to 65 lb. of steel per 100 sq. ft., gives excellent results in freedom from transverse cracks.

Joints as a means of decreasing cracks sometimes seem objectionable because of poor work often done at those points, of weakness that develops unless they are doweled together, and of delay to construction. To offset these disadvantages the State of North Carolina has built a test section of pavement in which planes of weakness are introduced every 40 ft., so that the slab will crack and divide itself, thus dispensing with joints. The steel reinforcement is omitted for a width of 1 ft. transversely across the pavement. After "belting", a battered wood strip, of ½-in. top, and ¼-in. bottom, width, and 2-in. depth, is forced into the concrete flush with the surface where

* Report, Highway Research Board, National Research Council, Vol. II, 1926.

the steel has been omitted. The surface at the strip is finished smooth with a trowel. After the initial set the board is pulled out, leaving a slot in the concrete which is filled with tar after the curing.

This method permits the speeding up of construction. It gives an interlock at the joint which will act as long as it is closely held together. Without steel, however, it tends to be forced farther and farther apart as time goes on. The main disadvantage of this method is that it does not provide for subsequent expansion, which, according to many authorities, is cumulative. If room for expansion is not provided in a longitudinal direction, ultimately there will be crushing of the concrete at the cracks, or diagonal cracking throughout the slab, where the accumulated stress from elongation is greater than the weakest parts of the slab. This work in North Carolina was constructed during March, 1925. During the curing period a crack formed, as was expected, at each plane of weakness.

An inspection in October, 1925, on a cool, cloudy day, showed open cracks at all the planes of weakness and no other transverse cracks in the 2 000 lin. ft. of pavement under the test. This is the only method yet devised of securing regular breaks in a pavement in which it is desired to omit the regular joints during construction.

Another recent development is the two-course pavement, which is again coming into general use as an outgrowth of sidewalk construction. This type has been proved to be a success by the wonderful service given by many of the old Blome concrete pavements. It has been used extensively, especially in the environs of Buffalo, N. Y., where there is a scarcity of good materials for concrete roads and pavements except at large expense for importation. Since 1922 more than 1 000 000 sq. yd. of this type has been constructed as follows: Total thickness, 8 in., the bottom 6 in. being of 1:2:4 concrete with gravel, stone, or slag aggregates; while the top 2 in. is of 1:1.5:2 concrete, using gravel up to $\frac{5}{8}$ in. in size; all other features were those usual to good construction. In some localities where good aggregates for the top are not available, a mortar top has been used with excellent results. Economically this type has the advantage of low cost on account of making poorer materials available for use. The fine aggregate or mortar top decreases materially the cost of surface finishing, and the richer mixture in the top surface adds materially to the resistance to abrasion, as has been shown by a number of investigators. This resistance has come to be a considerable factor where tire chains concentrate wear for many days at a time in ruts made through the snow.

Chapel Street, New Haven, Conn., is an excellent example of this type. This pavement, built in 1908, was of the Blome Granitoid type, more than 3 000 ft. long, on a main thoroughfare from the east, and probably the heaviest traveled street in the city. The pavement is $6\frac{3}{4}$ in. thick, the lower 5 in. being a 1:3:4 concrete, covered by a $1\frac{3}{4}$ in. top, composed of 1 part cement to $1\frac{1}{2}$ parts of clean crushed trap-rock having a maximum size of $\frac{1}{4}$ in. with the dust removed. The original markings are still in evidence after seventeen years, and the pavement is entirely serviceable, which is remarkable considering the number of steel-tired coal trucks that have passed over it.

Another illustration is a rural highway built under the writer's direction at Finley Lake, N. Y. This pavement was 4.33 miles long, with a 28-ft. roadway; the concrete surfacing, 10 ft. wide, and 6 in. thick, has a 4-in. base of 1:2:4 gravel concrete and a 2-in top of 1:2:3 imported stone. After seven years it is still in excellent condition.

An interesting invention has been made by E. G. Hooper, Assoc. M. Am. Soc. C. E., to meet the difficulties of uncertain conditions in drainage and subsoil. Where these factors are indeterminate, the logical plan is to eliminate the roadbed as a supporting medium, and predicate the design on artificially established supports for the pavement. Thus, a 20-ft. pavement is made up of two 10-ft. lanes of equilateral triangular slabs, each slab being supported by a pile at each of its three apices (corners). Each support carries the end of one or more adjacent units. Adjacent edges are designed to interlock to make adjoining slabs mutually supporting so that settlement of any corner will cause no appreciable change in design stresses. As the supports are carried to a solid foundation and below frost, they may be considered to be permanent. The moments and shears for the given loads may be calculated. Confirmation of the validity of the method of design has already been obtained through the use of test models. This type of pavement offers a promising solution for viaducts or for heavy fill sections over unstable soil.

On the theory of design based on the tests mentioned, a pavement thickness of 7 in. with 27 lb. of steel per sq. yd. is found adequate to carry the 12-ton axle load often specified for a 15-ton loaded truck. The cost of the pavement proper per unit of material used is not greater than for the usual pavement.

Among the most important of all recent developments is the application to practice of the researches* of Duff A. Abrams, M. Am. Soc. C. E. These researches definitely show that an increase in the water-ratio (wetter concrete), a decrease of cement, an excess of fine aggregate, or a poor grading, mixing, or curing, diminish the compressive and bond strengths of the concrete, and lessen its impermeability and its resistance to wear and destructive agencies. Had the results of this work been available ten years ago there would have been saved large sums invested in concrete that is already showing signs of limited life.

Every lad knows that he can make synthetic stone by mixing sand, gravel, cement, and water, and some concrete structures built with no more skill have miraculously survived. However, with a steadily increasing public demand that all sorts of structures beside pavements shall be of concrete, it behooves engineers to safeguard public funds by improving the knowledge of the scientific details of concrete work to the limit of their ability. Recently, $2 000 was wasted in one small town because a shoulder extension to a concrete pavement was laid flush to the curb without a particle of allowance for expansion; and the engineer in charge of the job was blissfully complacent!

Concrete of high early strength is coming more and more into demand for streets under heavy traffic, for emergency work, for replacements, bridge floors, cold-weather work, and, in general, wherever it is necessary to shorten

* *Bulletins Nos. 1–17,* Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

the period of curing. Properly controlled, 1:2.5:4.5 concrete, with 1.32 bbl. of cement per cu. yd. and 7.8 gal. of water per sack, will give a compressive strength of 750 lb. in 3 days. Changing the mix to 1:1.5:2.5, the quantity of water to 4.4 gal. per sack, and increasing the time of mixing to 5 min. will give a concrete of a compressive strength of 2 860 lb. in 3 days, which is almost four times as strong as the usual mix at that age. If still further acceleration is required, or if only the usual mix can be used, high early strength may be attained by the admixture of calcium chloride. This recent development is coming into general use and giving satisfaction. It should be safeguarded, however, by observing in a preliminary test to just what extent the calcium chloride hastens the set of the particular brand of cement to be used.

Important for safety is the increasing tendency to straighten alignments and build wider pavements. Engineers have advocated this for years; people are just beginning to demand it, and every effort should be made to keep before them the necessity for the jump from the 20 to the 40-ft. pavement in congested areas. The 30-ft. or 3-lane width on busy thoroughfares is a snare, especially if the traffic is evenly distributed both ways. That middle strip invites two cars to turn out at once and come head on to each other. With 20 000 000 cars now in use in the United States, and 3 000 000 more being built this year (1926), the public must realize, and engineers must help make them realize, that adequate roads mean not only safeguarding life, but also actual money saving in time, gasoline, and nervous energy.

Such are some of the important recent developments in concrete road building. Most important, perhaps, is the progress in knowledge and education in regard to all types of highway construction. For their large part in this progress tribute should be paid to those organizations which have made unfailing efforts to attain and maintain standards: The U. S. Bureau of Public Roads, the Asphalt Association, the Portland Cement Association, the Brick Association, the various engineering societies, and the American Society for Testing Materials. Upon them, and upon the engineers on the jobs, depends the success of American roads.

DISCUSSION

A. T. GOLDBECK,* ASSOC. M. AM. SOC. C. E. (by letter).—Periodically it is well to pause in any research undertaking, to look back over the established facts, note the general indications, and decide which conclusions are ready for practical application. Then the next step in advance may be taken more safely. Mr. Breed has recognized this necessity in his excellent review of the field of highway research as it applies to concrete pavement design.

Cross-Section Design.—Up to the present researches seem to have led to a theory of concrete road design which is based on a few basic facts, or at least what seem to be facts in the present state of knowledge. A high percentage of the State highway departments have accepted a design with thickened edges, and this general design seems to be well substantiated by field research and by the service results thus far obtained.

The theory on which this so widely accepted cross-section is based is elementary. It is premised in general on the following ideas:

1.—Traffic uses the entire pavement area and the pavement must be capable of supporting heavy wheel loads wherever applied.

2.—It is particularly undesirable that the pavement be cracked into slabs of small area, for rapid subsequent failure and high maintenance expense will ensue.

3.—Since a corner crack produces a small area of slab this is an undesirable condition which should be prevented.

4.—The structural strength of the concrete alone can be relied on for supporting a load applied at the corner of the slab and sub-grade support must be neglected.

5.—Concrete suffers fatigue rapidly when the bending stress in tension exceeds one-half the modulus of rupture of the concrete.

These premises, as has been pointed out in detail a number of times, lead to the design formula for edge thickness:

$$D = \sqrt{\frac{3\,P}{S}} \qquad (1)$$

in which,

D = edge thickness of pavement.
P = maximum wheel load.
S = allowable unit stress in concrete in tension which should not exceed one-half the modulus of rupture.

When a center longitudinal joint is used, of such a design that the load is carried by two slab corners instead of one, Equation (1) is applicable for calculating the center thickness, d, as follows:

$$d = \sqrt{\frac{3\,\frac{P}{2}}{S}} = \sqrt{\frac{1}{2}}\sqrt{\frac{3\,P}{S}} = \frac{7}{10}D \qquad (2)$$

* Director, Bureau of Eng., National Crushed Stone Assoc., Washington, D. C.

It is to be observed that the pavement thicknesses at edge and center, calculated in the foregoing manner, are based entirely on the necessity for preventing small slabs from forming. Moreover, the possibility of high bending stresses occurring except at the corners of pavement slabs is not accounted for in the theory given. Deformation readings taken by the U. S. Bureau of Public Roads show that the highest bending stress occurs along the edge of the slab directly under a wheel load placed at the edge. When it is remembered that this bending stress must be combined with direct tension produced by shrinkage from drying or contraction from temperature decrease, it will be seen that much higher longitudinal tensile stress will be produced in the slab than is produced by bending at the corner.

This fact, however, need not lead to the conclusion that the corner theory of design is not adequate for determining pavement thickness. It may mean, however, that with this theory a somewhat lower unit stress should be used than one-half the modulus of rupture. Another important fact is that the fatigue tests on which the selection of the unit stress has been based, were conducted on concrete in a dry condition; some tests are available showing that greater fatigue takes place in wet or damp concrete than in dry concrete. In other words, a still lower unit stress might be desirable to take account of this fact.

The net result is that, to be safe against undue cracking of concrete roads after a period of years, a unit stress of less than one-half the modulus of rupture should be used. To offset this there is the further fact that a high percentage of wheel loads is applied from 1 ft. to 2 ft. from the edge of the slab rather than at the edge, and the edge stress thus only reaches its maximum value occasionally. The question of unit stress, however, bears further watching and further study.

Reinforcing Steel.—It seems to be quite well established that the usefulness of reinforcing steel in concrete roads lies not so much in its effect on the resisting moment of the concrete, but rather in holding the concrete in close contact after a crack forms. Under certain conditions it is likewise effective in reducing the amount of cracking.* To hold the crack together, it is necessary that the steel be proportioned so that it will not be stressed beyond the yield point. A method for calculating the amount of steel necessary to accomplish this purpose has been described in detail elsewhere.†

The formula proposed for this calculation is:

$$f \frac{L}{2} W B = a S_1 \qquad (3)$$

in which,

L = spacing of transverse joint, in feet.
B = width of pavement, in feet.
S_1 = allowable tension in steel, in pounds per square inch.

* "Report of the Investigation of the Economic Value of Reinforcement in Concrete Roads," by C. A. Hogentogler, December, 1925, Annual Meeting, Highway Research Board, National Research Council.

† "The Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads," by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., *Public Roads*, August, 1925.

f = coefficient of sub-grade friction.
W = weight of concrete, in pounds per square foot.
a = cross-sectional area of longitudinal steel.

Equation (3) simply states the obvious fact that the tension in the steel at a crack must equal the total sub-grade friction between that crack and a free end of the slab, such as exists at an expansion joint. It also brings out clearly that the amount of longitudinal steel which should be used depends on the spacing of expansion or construction joints.

The amount of transverse steel might be calculated by the same formula. The transverse steel should be capable of sliding the slab transversely over the sub-grade without overstressing the steel. Theoretically, this would require the largest amount of transverse steel at the center and none at the edge. Similarly, the longitudinal steel should be of greatest cross-section midway between expansion or construction joints and could taper to nothing at the joints if this were practicable. The value for f in Equation (3) might be as much as 2.0, but is generally lower than this amount.

Equations (1) and (3) form the basis for a simple theory for the design of concrete pavements.

The Sub-Grade.—Much has been written about the effect of the sub-grade. Thus far, however, the different sub-grade characteristics are not recognized in the cross-section design. It is known that the sub-grade does exert an important influence on the behavior of the road and in certain cases, as in the worst soils, a granular blanket layer seems to be of value.* Much more research is needed on the sub-grade and this should now take the form of correlation of field behavior and sub-grade characteristics. Until this is done the science of the sub-grade cannot advance properly. The laboratory procedure of soil analysis, however, has been developed to a satisfactory degree, and it is to be hoped that means will be forthcoming for carrying the field investigations to a practical conclusion. This will give a definite knowledge of how the pavement design should be changed for soils having different characteristics.

Research has developed certain fundamental facts now combined into theories which, although not entirely rational, are at least resulting in serviceable roads. Large mileages of concrete roads remain to be built in the future and a number of details of design, construction, and materials, and the economics thereof, need further investigation. Engineers cannot afford to lie back now in their research work, complacent with the thought that they have sufficient information.

Elmer G. Hooper,† Assoc. M. Am. Soc. C. E.—The question of the triangular slab pavement has been raised. The rigid-slab type of pavement as usually laid on a natural soil sub-grade is not susceptible of exact analysis for design. It is an indeterminate structure and the important unknown factor is the support. The condition and character of the supporting soil in many cases may influence the amount and distribution of stresses in the pavement only slightly, but a poor sub-grade presents a serious problem. The

* "The Present Status of Subgrade Studies," by A. C. Rose, Assoc. M. Am. Soc. C. E., *Public Roads*, September, 1925.

† Asst. Prof., Civ. Eng., New York Univ., New York, N. Y.

structural stresses in the pavement are reversible and become excessively large for economical design. If a pavement can be built in which the units rest on permanent supports at known points, it provides a structure the behavior of which can be predicted from analysis or experiment. Any type of supported slab pavement that offers simplicity in design, reasonable ease of construction, and moderate cost is preferable to the indeterminate slab on a poor base and should find an extensive field of use.

A pavement made up of triangularly shaped slabs supported at their corners was conceived as satisfying the conditions enumerated. The three-point support of units assures definite stresses with no reversal even if the supports settle unevenly. The shape and economical size admit of precasting, transporting, and assembling the units in the pavement expeditiously. It is possible to remove slabs from the pavement for replacement or salvage.

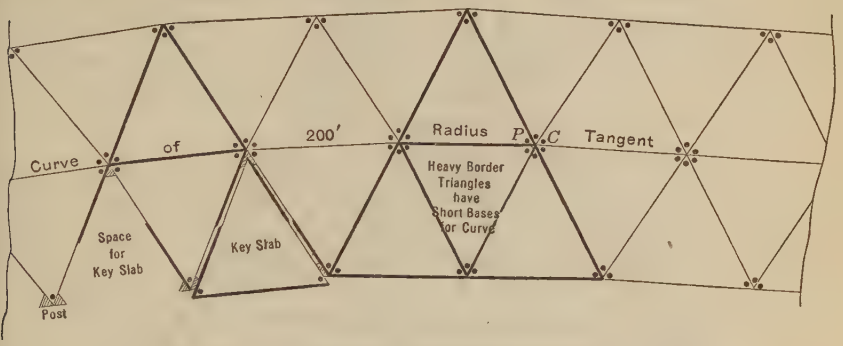

Fig 1.—Triangle Highway Pavement, Assembly of Slabs.

The triangular units are assembled as shown by the plan for a two-lane road in Fig. 1. Each lane is made up of a row of triangles placed edge to edge, their altitudes representing the lane width and the bases forming alternately the left and the right side of that lane. In this manner three slabs come together for support on a single pile or post. Where two lanes are contiguous the supporting posts under the line of contact carry the corners of six slabs each. Contiguous edges of slabs are provided with special interlocking lugs and recesses so that deflections and stresses in one may be shared by its neighbors. Loosely fitting pins through the vertices of the triangles into the posts prevent undue lateral movement after assembly.

The triangular slab is as nearly determinate as any plane surface or slab. Loads in any position are bound to come on or within the three lines joining the supports so that the deflection is convex downward, producing tension in the bottom and compression in the top of the slab. The direction and magnitude of these stresses can be determined because service conditions can be duplicated for test purposes. Studies already made have shown the behavior of steel plate and of reinforced concrete models of one-third scale. The results make it possible to proportion and place the steel to take the tension in the triangles. Fig. 2 shows an amount and arrangement of steel in a 7-in. slab for a pavement intended to carry loads up to 12 tons on one axle.

It is interesting to note some outstanding results obtained from tests. The greatest bending stress was developed when the load was placed at the middle of one edge. Failure from a load in this position occurred as a break beginning under the load and extending across to the opposite apex, Fig. 3 (*a*). The slab can be conceived as made up of a number of strips, or beams, of unit width, stuck together and shaped like a triangle by making an outside strip serve as the base and sawing diagonally across the grain for the other sides. With this conception the slab failure mentioned corresponds to the breaking of the whole series of beams at their centers, Fig. 3 (*b*). The tests showed

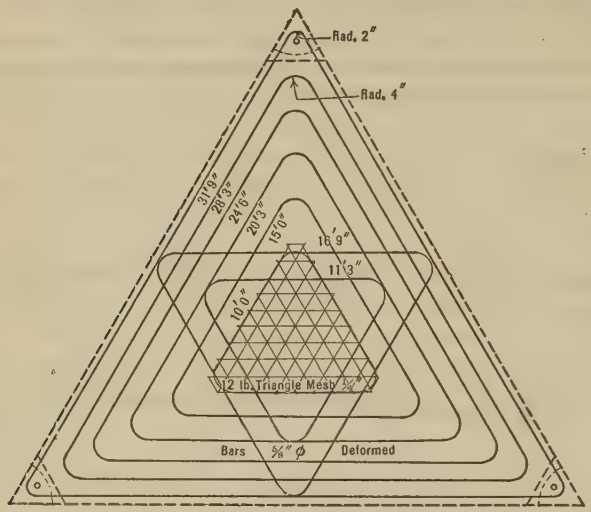

FIG. 2.—STEEL ASSEMBLY PLAN, TRIANGULAR HIGHWAY PAVEMENT UNIT.

conclusively that the whole cross-section represented by the altitude of the triangle was bearing the load, and the deflection of each of the unit strips, considered as a simple beam, was an index of its part of the whole load. The average load per unit strip is the total load divided by the altitude of the triangle. The deflections measured across the altitude show that the strips do not carry equal parts, instead there is a maximum at the load reducing uniformly to zero at the apex. Directly under a load on the edge the deflection corresponds to a simple beam deflection under twice the average unit load. For example, a triangle with a base length of 12 in. and an altitude of 10 in., and bearing a load of 1 000 lb., would produce an average load of 100 lb. per in. of altitude, whereas the deflection under the middle of the 12-in. base would be the same as that produced in a simple beam by 200 lb.

Shearing stresses are naturally most important at the apices of the slabs. The critical cross-section to resist shear depends on how far the support extends under the corner, the larger bearing giving the greater shear section and, accordingly, the lesser unit shearing stresses. In order to increase the bearing and improve contact between the slab and its support, it is proposed to use cushion blocks of treated wood, that shall extend their own thickness

beyond the edges of the posts. The necessary shear-resisting area may be obtained in this way to permit the use of an allowable unit stress suited to the type of reinforcement without stirrups.

The triangular slab resting on three supports placed under its apices has advantages over slabs supported at four or more points. In the case of the latter a small settlement of one support would result in a "teetering" slab in which the stresses would become complex and excessive. The triangular slabs would show the settlement as an unevenness of the pavement without developing any appreciable change in stress distribution, and the low point can be raised and "shimmed" when it is found convenient to do so. The triangular shape lends itself more readily to handling as precast units because interlocking edges, (Fig. 3 (*a*)), can be constructed so that the units may be assembled in the pavement as a whole or any unit may be removed and replaced by the simple expedient of removing the pins that keep the corners from sliding off the posts and then moving the slab laterally about 6 in. to free it from its lock with adjoining slabs (Fig. 1).

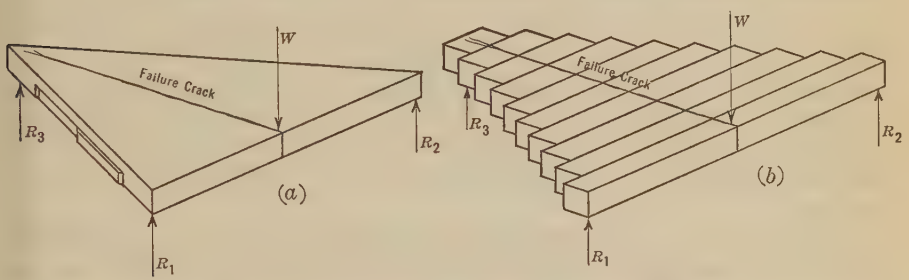

FIG. 3.—TEST RESULTS, TRIANGLE HIGHWAY PAVEMENT.

Expansion must be provided for in this type of construction as in any other. The joints are far more frequent than in the usual type, therefore the space to be allowed at each one is correspondingly smaller. Slabs of normal size provide two diagonal joints every 12 ft., an average of one in 6 ft. Expansion for a range of temperature of 150° is about $\frac{1}{16}$ in. in the 6 ft. A $\frac{1}{2}$-in. strip of treated felt stuck to one of the faces of a joint before the contiguous slab is pushed into contact, should provide sufficient "go and come". In case this allowance is not sufficient the angle of contact, together with the slight play at the loose pins, will permit slabs to accommodate themselves by lateral movement.

There are normally two conditions for design. In the first case the subgrade is just free of the under side of the pavement, so that failure of a unit can cause no damage to traffic and a low factor of safety can be used in the design. In the second case the pavement is far enough above the ground to be considered as a viaduct or bridge in which a slab failure might have more serious consequences and, therefore, a larger factor is desirable and, in addition, a curb about 1 ft. high integral with the slab is necessary to form a guard.

If in the future heavier loadings are desirable for any highways in which the triangular type of pavement has been used, it is feasible to salvage the slabs and use them again where the original load limits still prevail.

T. M. Ripley,* M. Am. Soc. C. E.—Mr. Breed has mentioned the advisability of constructing a road on piles. There are in New York State soils of almost every known character from muck and swamp to granite mountains, yet in none of these is it possible to visualize a road on piles unless it becomes a bridge.

Unless Mr. Breed is misunderstood, this pavement on piles can be built as cheap, or more cheaply, than some other kinds. Where it can be so built it must be marsh, swamp, or open water, and it seems that in any of these locations such a structure would take more the attributes of a bridge than of a highway as the terms are commonly used.

Although the writer has had considerable experience in building and maintaining roads for the State of New York, nevertheless the road on piles is difficult to visualize and more definite information is necessary, in order to understand where it would be economical to build such a pavement.

W. C. Hammatt,† M. Am. Soc. C. E. (by letter).—One phase of concrete slab construction has received little attention in the past. The proper proportioning of concrete mixes and the proper mix to produce the greatest strength are determined, the necessary tests being made under laboratory conditions to confirm the assumptions. This practice is now well established, but the reproduction of the results under field conditions is not so far advanced.

A comparatively recent development in California along these lines may be cited. Over large areas, particularly in the San Joaquin Valley and in Southern California, the prevailing subsoil is sandy with a high capillarity. The customary practice was to prepare the sub-grade by leveling and rolling and to wet it thoroughly just prior to laying the concrete slab. Thereafter, the slab was diked in and flooded for a period of several days for curing under water. This produced a well-cured upper surface, but a slab varying in strength from its upper to its lower face, since the tendency of the sandy subsoil was to rob the mix of its water before the final set was completed.

To overcome this condition a process has been devised by which a waterproof skin is interposed between the sub-grade and the pavement. By this means the water content of the mix is kept constant throughout the setting period and the strength is kept uniform throughout the depth of the slab. In other words, laboratory conditions are reproduced by placing the concrete in a water-tight form.

The same process has been found efficacious for use on a clay sub-grade. It was previously considered that the only way to prevent cracking of the slab over a clay sub-grade was by means of copious reinforcement, but this idea was fallacious. The reason for the cracking was that, on saturation, the clay subsoil swells. In placing the concrete pavement thereon it is to a cer-

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† Cons. Engr., San Francisco, Calif.

tain extent bonded to the clay subsoil by the penetration of grout into the saturated soil. With the subsequent drying and shrinkage of the subsoil a tension is set up in the concrete slab, which, when accompanied by the impact and vibration of passing vehicles, soon causes rupture.

In Fig. 4, let *A B* represent a concrete bar subjected to opposite forces at *A* and *B*. The tensile stresses created may be well within the strength of the bar, but if it is subjected to even a light, but constant, tapping at *C*, the vibration will sooner or later produce rupture at that point.

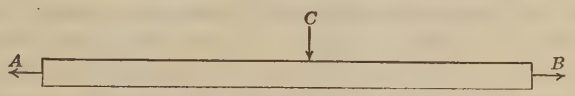

FIG. 4.

The interposing of the water-proof skin, previously mentioned, between the sub-grade and slab serves two purposes in this case. It obviates the necessity of wetting the sub-grade, thus permitting it to be compressed dry so that there will be no subsequent shrinkage and no consequent tensile stresses set up in the concrete slab. It also prevents the bonding of the slab to the sub-grade and produces the condition of a precast slab.

The former practice was to interpose a layer of sand or gravel between the clay sub-grade and the concrete slab, but this method had obvious disadvantages. Although, theoretically, it forms a cushion preventing the transfer of tensile forces from the clay subsoil to the concrete slab, in practice the grout from the bottom of the pavement mix gets into the top of the sand course producing a pavement weak at the bottom and bonded to the cushion course.

Some contractors in Central and Southern California are using a new method, known as the "Monolite Process", for all concrete pavements regardless of subsoil conditions, although it is the writer's belief that its merit attaches only to the soil conditions mentioned, and possibly also to alkaline conditions.

R. W. CRUM,* M. AM. SOC. C. E.—A new form of pavement (Fig. 5) has been much used in the rougher sections of Iowa. This is a curb and gutter section designed especially to carry the surface water on the slab instead of on the shoulders and in side ditches, but it has other advantages. Structurally, it

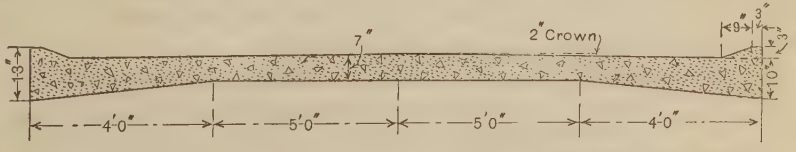

FIG. 5.—PAVEMENT SECTION USED BY IOWA HIGHWAY COMMISSION ON HILLS TO CARRY SURFACE WATER ON SLAB.

has a thickened edge; it tends to keep the wheel load at least 12 in. from the edge of the slab; and the shoulder is not softened by run-off water from the

* Engr. of Materials and Tests, Iowa Highway Comm., Ames, Iowa.

slab. There is also no obstruction to the wheel running off the slab on to the shoulder at almost any speed.

This slab section itself does not cost materially more to build than the customary thickened edge section, but the intake structures necessary for the removal of the water do increase the cost of the road as a whole.

CARLTON N. CONNER,* ASSOC. M. AM. SOC. C. E. (by letter).—Since the results of the Bates Road tests have been quite generally accepted there has been a feeling that no new developments in concrete pavement had been made, but Mr. Breed's paper and a study of the situation show that progress in construction methods and design detail of far-reaching importance are still going on. The author has described some of these and has touched on others.

The use of steel reinforcement consisting of small steel members closely spaced is being studied by the State of Virginia in conjunction with the U. S. Bureau of Public Roads. Although the report of the Advisory Board on Highway Research of National Research Council indicated that progressive destruction of pavements is retarded by the use of reinforcement, tests are now going on which indicate at least that initial cracking, due to shrinkage before the final set of the concrete, is reduced by the use of reinforcement.

The author refers to a test section of pavement in North Carolina in which transverse planes of weakness or "dummy" joints were introduced every 40 ft. The writer was responsible for this test and found during an inspection of this same pavement made in September, 1926, that only one transverse and no longitudinal cracks in the 2 000 ft. of test section had developed, except at the planes of weakness. The 1926 inspection was made 18 months after the pavement was laid.

Since 1924, however, this method of introducing longitudinal planes of weakness along the center line of pavement has been used in several other localities with satisfactory results. Delaware was one of the first States to use this method of controlling and localizing longitudinal cracks. Other States have also used it, including Pennsylvania, Michigan, and California, as well as the City of Seattle, Wash. For those who feel that the cost of joints is not justified this method promises to become an economical and satisfactory substitute.

More uniform strength and greater plasticity of pavement concrete is being secured by inundating the sand so as to obtain an amount equivalent in volume to dry sand. This method of proportioning has also resulted in a saving of cement without a sacrifice in strength of concrete.

The use of calcium chloride in the mix rather than on the surface for curing and a high early strength seems to have gained considerable attention. Its use on the surface although prevalent at one time in some States has been practically discontinued.

Although for years testing engineers have felt that the results of the compressive tests of cylinders and cores taken from roadways were often inconsistent and unsatisfactory, yet it was only during 1926 that any other method

* Chairman, Investigation of Low Cost of Improved Roads, Highway Research Board, National Research Council, Washington, D. C.

of determining the strength of roadway concrete had been practically demonstrated as a field test under field conditions. This is the test in flexure of small beams cast from the roadway concrete, cured alongside the road, and broken at the roadside.

A cantilever load is applied to the beam in a suitable and simple machine. The results have been satisfactory and much more uniform than were ever obtained with compression tests of cylinders and cores. The stress due to flexure or cross-bending more closely approximates that in a concrete pavement than compression alone. By using this simple and practical method it is now possible to determine the number of days before a pavement should be opened to traffic with a greater degree of accuracy.

All these satisfactory results and methods have been the result of practical research for facts, which tends toward progress.

H. C. Boyden,* M. Am. Soc. C. E.—For many years there has been much discussion regarding the need for reinforcement in rural concrete pavements. Recent observations indicate that it pays from all points of view to use reinforcement of 60 lb., or more, per 100 sq. ft., and that the best results are obtained when the steel is in small, rather than large, units. This observation favors wire mesh at the expense of bars, but it also implies that the mesh shall have been properly placed.

Many engineers have objected to mesh because of the difficulties attendant on placing it properly in the pavement. To-day, however, with mesh shipped in flat sheets rather than in rolls, or with proper straighteners on the job when the material is shipped rolled, there should be no greater difficulties than when bars are used.

Reinforcement does not add materially to the structural strength of the slab; but it does prevent some cracks, holds together such cracks as do occur, and distributes impact from moving loads over greater areas.

C. Raymond Hulsart,† Assoc. M. Am. Soc. C. E.—The importance of early strength in concrete warrants further details than the author has given.

Certain field conditions sometimes make it difficult to secure a high early strength with standard Portland cement. For instance, the small quantity of mixing water, 4.4 gal. per bag of cement, will be difficult to use with many commercial aggregates. Also, the long-time mixing may be difficult or expensive to secure, in that it would cut the contractor's mixer output to about one-third the normal. In using calcium chloride, mixed with the concrete, the engineer should have in mind that it re-acts differently with different brands of Portland cement and does not re-act at all with some brands. Tests with the brand of cement to be used will determine the exact quantity of calcium chloride required for the desired effect. Further, the proportioning and mixing with the calcium chloride is an accurate process, requiring careful attention.

This difficulty has encouraged the development of special quick-hardening cements, or early strength cements, especially in Europe. There, the so-called

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† Vice-Pres., The Atlas Lumnite Cement Co., New York, N. Y.

"super-Portlands" or "super-cements", which have quick-hardening properties, are generally secured by finer grinding and a high lime content. Even in Europe considerable difference of opinion exists as to their permanence and durability. These cements have not, as yet, been manufactured in America.

The quick-hardening cement longest in use in Europe (since 1912, principally in France) is not a Portland cement, but an alumina cement made from aluminum ore (bauxite). The product is known as "Ciment Fondu". Previous to the World War, its use was principally on account of its resistance to the chemical attack of sea water. During the war it was used for the construction of heavy artillery foundations which could then be put in service in 24 hours. This gave aluminate cements an impetus which has brought about their rather general use throughout Europe.

By high early strength is meant that within 24 hours they become considerably stronger than other cements in 28 days. Aluminate cement known as lumnite has been used in America since the summer of 1924 to meet special conditions, such as intersections of roads and streets, or for certain short pieces of road where traffic could not long be interrupted. For example, State highway departments have used aluminate cement for important intersections that have been poured one day and thrown open to traffic the next. Many highway departments have used aluminate cement also for the repair of concrete pavements and for replacements.

By way of illustration, in Maryland, a road was to be constructed leading to a pass under a railway. There were banks on each side; the traffic could not be detoured conveniently, and it was so heavy that it could not be stopped for any considerable length of time. The State Highway Department constructed a section of pavement with aluminate cement, finishing it on one afternoon and on the next throwing it open to heavy motor traffic.

Likewise, in New York City, at Lafayette and Houston Streets, according to the Chief Engineer of the Borough on the heaviest continuous traffic artery in the world, a section of the street was closed on Friday night, the concrete base was placed with aluminate cement, the granite block put on top of it, and the road was thrown open to traffic on Monday morning.

These aluminate cements, due to their rapid chemical action in hardening, develop considerable heat, so much so that they have been found safe for use in cold weather. For example, near Netcong, N. J., there was 1 200 ft. of road under construction and unpaved in December, 1924. It was not considered desirable to leave it in that state through the winter. This stretch was finished on Saturday night in freezing weather, using aluminate cement, and was thrown open to traffic on Monday morning.

Edward Godfrey,* M. Am. Soc. C. E. (by letter).—Mr. Breed's paper offers evidence of the fact that the problem of the best way to lay concrete pavements has not been determined or at least agreed upon. The matter of shrinkage cracks and their avoidance has always been a problem difficult of solution. One of the standard methods of building concrete sidewalks is instructive on this point. A perfect bond between the base and the finish

* Structural Engr., Pittsburgh, Pa.

coat may be obtained by using mixtures having mortar of the same richness for each layer, and by placing the upper course before the lower has set. This avoids any possible unequal shrinkage such as often takes place in the case of mortars of different richness. This danger is also an argument against two-course construction in road work unless the second course is placed immediately after the first one is poured.

Instead of being laid in long lengths at one time, the pavement might well be blocked off into sections and alternate slabs poured first, followed by the intermediate sections. In this way one-half the shrinkage of setting is eliminated entirely, and the shrinkage cracks, if any occur, are definitely located at the construction joints.

In the ordinary way, where long lengths of road paving are placed without break, except by the introduction of separators or by the questionable method of driving in a plate to cut up the concrete, it cannot be expected that the best results will be obtained. When one or more squares or oblong blocks of concrete are poured against bulkheads and allowed to set, all the shrinkage of those blocks is taken up. Then when the bulkheads are removed and the intermediate blocks are poured, two things are accomplished: First, the longitudinal shrinkage of setting is only that of the second set of blocks; and, second, there is little adhesion between blocks, although a perfect fit, and any tendency toward shrinkage cracks is forced to these construction joints.

Mr. Breed makes the statement that, according to tests made by Professor Abrams, wetter concrete, among other things, means more permeable concrete. He intimates that compressive strength and impermeability vary with diminished quantity of water in the mix, and that the former quality is a measure of the latter. This is a very commonly held notion. It is, however, not shown by Abrams' tests to be true.

The tests made by Abrams* are on absorption and not on permeability. There is an essential difference. Permeability allows the passage of water through the mass, whereas absorption is a measure of the water that enters the pores of the mass. A cork will absorb water but will not let it pass through the mass. A sound cork will absorb more than 50% of its weight of water. A spongy mass of iron may be quite non-absorbent, but may allow the passage of water freely. Much concrete will allow the passage of water freely, although it may not be particularly absorbent. The writer has seen concrete that had large spaces between the stones, but had the appearance of a non-absorbent concrete. The type of dry, tamped concrete that was universally used nearly thirty years ago was famous for being able to allow water to flow through thicknesses as great as 30 ft. It is to be noted that this is the zone where Abrams finds the greatest strength.† It is certain that this concrete would be neither non-absorbent nor impermeable.

Abrams' tests on absorption are few in number for a determination of so great importance, especially when compared with the hundreds of thousands

* *Bulletin No. 2*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

† *Loc. cit.*, p. 6.

of tests on the less important property of compressive strength. For reinforced concrete work a property of concrete of greater importance than compressive strength is impermeability, for the permanence of reinforced concrete would be jeopardized, if the concrete itself were not impermeable and did not protect the steel.

As stated, Abrams' absorption tests do not cover the zone of high compressive strength, as the water ratio extends only to 0.66 as a lower limit, and the strength curve takes a sharp upward turn below this ratio. Besides, while the specimens not kept wet during curing show a strong upward trend in absorption in the wetter mixes, the specimens stored in damp sand do not show similar results. Furthermore, there are comparatively many examples where the specimens cured in damp sand show much less absorption in the wet mixtures than in the dryer ones. The specimens cured in damp sand are really the properly treated ones, and their showing is of much greater value than the others. These indicate a trend toward greater absorption both in the wetter and the dryer mixes and a minimum absorption at water ratios between 0.73 and 0.91, which is the zone of average strength and consistency (and includes freely flowing concrete) and not that of high strength.

MEMOIRS OF DECEASED MEMBERS

MORDECAI THOMAS ENDICOTT, Past-President, Am. Soc. C. E.*

DIED MARCH 5, 1926.

This memoir records the professional career of a Past-President of the Society, who was closely associated with Naval shore construction activities for more than forty years, and who was also an authority on Isthmian Canal projects.

Mordecai Thomas Endicott was born at Mays Landing, N. J., on November 26, 1844, the son of Thomas Doughty and Ann (Pennington) Endicott. He was a direct descendant of John Endicott, the first Governor, in 1629, of the Massachusetts Bay Colony. He received his primary education in the schools of the Presbyterian Church and through private instruction at Mays Landing, N. J. In 1864 he entered the Rensselaer Polytechnic Institute, at Troy, N. Y., where he pursued the regular course in Civil Engineering, and from which he was graduated in 1868 with the degree of Civil Engineer.

Following graduation he entered actively upon his engineering career. He was first employed as an Office Assistant by the late R. P. Rothwell, M. Am. Soc. C. E., a Civil and Mining Engineer, of Wilkes-Barre, Pa., from July until December, 1868. He next served as a Rodman in the Brooklyn Navy Yard until January, 1870, when he entered the service of the New Haven, Middletown, and Willimantic Railroad Company as a Draftsman, with headquarters in New York, N. Y. After spending five months in the office he was detailed to the field as an Assistant on the construction of the foundations and approaches of the Connecticut River Bridge at Middletown, Conn. Mr. Endicott continued on this work until the latter part of 1870, when he accepted an appointment as Assistant Engineer in charge of the "Dresden Extension" of the Cincinnati and Muskingum Valley Railway, from Zanesville to Dresden, Ohio.

On the completion of this railroad extension, he was appointed, on February 1, 1872, in a civilian capacity as an Assistant Civil Engineer at the then newly established League Island Naval Station at Philadelphia, Pa., and continued thereafter in the Government employ. After rendering meritorious services for a period of eight months in the development of the new Naval Station and on the construction of public works and public utilities, he was given orders placing him in charge of the Civil Engineering Department of the Philadelphia Navy Yard, where he continued until he was commissioned a Civil Engineer in the Navy on July 13, 1874, and ordered to the Naval Station at New London, Conn.

Following five years of service in charge of the Civil Engineering Department at the Naval Station at New London, where he designed and constructed wharves, buildings, and other improvements, Mr. Endicott was ordered, in

* Memoir prepared by the following Committee: H. H. Rousseau, *Chairman*, and C. L. Strobel, Members, Am. Soc. C. E., and A. N. Talbot, Past-President, Am. Soc. C. E.

1879, to similar duty at the Navy Yard at Portsmouth, N. H. In 1881, he was sent to the League Island Navy Yard at Philadelphia, where he remained in charge of public works design and construction until 1886, when he was ordered to the Norfolk, Va., Navy Yard. Here, in addition to other work, he was in charge of the construction of the dry dock that was completed in 1889. While at Philadelphia, in 1882, he was given the rank of Commander.

In October, 1889, Commander Endicott was detached from duty at Norfolk, and appointed by the Navy Department as a member of a Board to consider and investigate conditions and necessities at the New York Navy Yard, and to report on a plan for its development and improvement. On completion of this Board duty in April, 1890, he was detailed as Consulting Engineer to the Bureau of Yards and Docks, United States Navy Department, in Washington, D. C., and placed in charge of all civil engineering work under the cognizance of the Bureau.

This duty opened up a unique opportunity for him, for, while the construction of modern naval vessels had begun, the Navy Yards had not yet emerged from the condition in which they were when wooden vessels were supreme. Carts drawn by oxen still met transportation needs. Electric installations for light and power had not been introduced. Street improvements were principally noticeable by their absence. Lumber was the material used for dry-dock construction, and brick and lumber for buildings. The day of steel and concrete for Navy Yard construction had not arrived. How well Commander Endicott availed himself of this opportunity to modernize the Navy Yards may be judged from the record of his subsequent career and advancement, during the sixteen years that passed while he was on active duty. During this period, 1890-1906, there was also a tremendous growth and advancement in all lines of engineering and industrial accomplishment and activity outside the Government service, and it was his constant aim that the Navy should keep up with the pace set by the country's industrial plants and establishments.

Commander Endicott, then 46 years of age, was eminently fitted for this assignment at the Navy Department, having had eighteen years' experience at the principal Navy Yards along the Atlantic Coast, during which period he had become thoroughly familiar with naval conditions and requirements. This detail Commander Endicott held until his appointment as Chief of the Bureau of Yards and Docks, with the rank of Commodore, on April 4, 1898.

During this eight years of service as Consulting Engineer in the Bureau of Yards and Docks one hundred and ten construction contracts were entered into, embracing miscellaneous and varied engineering structures. Among the outstanding projects may be mentioned a timber dry dock for the Puget Sound Navy Yard, the design for which included a concrete and stone entrance, a marked improvement over the standards previously followed; turn-tables for 40-ton cranes; timber dry docks at New York and Port Royal, S. C., with improvements similar to those adopted for the dry dock at Puget Sound; and the design of 40-ton locomotive jib-cranes capable of handling the armor plates required by the battleships then being constructed. On March 21, 1898,

immediately prior to his appointment as Chief of Bureau, he was promoted to the rank of Captain, and by the provisions of the "Personnel" Act of Congress, effective March 3, 1899, he was elevated to the rank of Rear Admiral, which rank he held thereafter. He was the first Civil Engineer of the Navy to hold this rank.

Commander Endicott's reputation as an engineer, and his success in Naval civil engineering work led to his appointment by President Cleveland, on April 26, 1895, on the recommendation of the Secretary of the Navy, and without any application on his part, as the Navy member of the Nicaragua Canal Commission of 1895, headed by the late William Ludlow, Col., Corps of Engineers, U. S. Army, M. Am. Soc. C. E. The late Alfred Noble, Past-President, Am. Soc. C. E., was the third member. This Commission examined the route of the canal as proposed by the Maritime Canal Company, and reported on its feasibility, cost, and permanence. The Commission's report led Congress to delay taking action on bills furthering the Nicaragua Canal project which was eventually abandoned in favor of the Panama Canal route.

By Act of Congress approved July 19, 1897, the Navy was specifically forbidden by law to pay more than $300 per ton for armor for battleships; and if he could not purchase armor within this limit, the Secretary of the Navy was authorized to take steps to establish a Government factory of sufficient capacity to make such armor. The Act further directed the Secretary of the Navy to appoint an Armor Factory Board, to consist of competent naval officers, to advise and assist him in executing the authority conferred. Commander Endicott, then acting as Consulting Engineer to the Bureau of Yards and Docks, was selected as the Civil Engineer member of this Board. The report of the Board which went very fully into the design, construction, and the operation of a Government Armor Plant and which was accompanied by plans and estimates of cost, impressed Congress with the feasibility and economy of this project, and was largely responsible for the considerable reduction in the subsequent quotations for armor plate, as compared with the bids made prior thereto, as the armor-plate manufacturers then realized the Government's determination and ability to manufacture its own armor plate if they were unable to meet its views as to price.

In the spring of 1898, the four-year term of office of the Chief of the Bureau of Yards and Docks expired, and a new Chief of Bureau was to be appointed. This country was then on the eve of the Spanish War, and the new incumbent would have to carry the work of this important Bureau through a war the magnitude and duration of which could not be predicted. The man most familiar with the work of the Bureau and the shore needs of the Naval Establishment was Commander Endicott, then on duty in the Bureau. To appoint him Chief of Bureau would necessitate breaking the traditions and precedents of the preceding fifty-six years during which time only officers of the line of the Navy had held this position. The precedent was broken. President McKinley, on the recommendation of Secretary of the Navy John D. Long, submitted the name of Mordecai T. Endicott to the Senate for confirmation, and on April 4, 1898, he was appointed Chief of the Bureau of Yards and

Docks, the first officer of the Corps of Civil Engineers to be honored with this appointment. The success of his first four years of administration of this office resulted in his re-appointment in 1902 by President Roosevelt for another four-year term, and he was given a second re-appointment by President Roosevelt in 1906.

He reached the then retiring age of sixty-two years on November 26, 1906, but continued at the request of the Secretary of the Navy to serve as Chief of Bureau until he resigned this office on January 5, 1907. He remained on duty in the Navy Department until March, 1907, when he was detailed to the Department of Justice as Technical Adviser to the Attorney General of the United States on suits being prosecuted against the Navy Department. On June 30, 1909, on the completion of this work, he was relieved of active duty and ordered to his home.

A few of the outstanding accomplishments of Admiral Endicott's nearly nine years of service as Chief of the Bureau of Yards and Docks should be mentioned. When he was appointed, the Corps of Civil Engineers comprised only thirteen officers. This small Corps was inadequate in numbers to meet the rapidly expanding requirements of the Navy. Through the discretionary power to make additional appointments vested in President McKinley, the Corps was increased to eighteen; and, later, on March 3, 1903, through the efforts of Admiral Endicott, Congress fixed the number of officers in the Corps of Civil Engineers at forty. The successful conduct of the office of Chief of Bureau by an officer of the Corps of Civil Engineers had demonstrated the wisdom of this policy.

It is natural to assume that a technically trained man with administrative ability is better qualified to handle the work of a strictly engineering bureau than one who has to rely on assistants for the technical advice and information on which he has to make his decisions. Admiral Endicott's efforts were untiring to promote the welfare of the Bureau and the Corps by securing the adoption of this practice as a permanent policy before he gave up the Bureau, and he was finally successful in convincing Congress of the advisability of restricting future appointments to officers trained in civil engineering. A law passed in 1906 indicated Congressional approval of this policy, and required that thereafter the Chief of the Bureau of Yards and Docks should be appointed from the personnel of the Corps of Civil Engineers.

Admiral Endicott's name came prominently before the Engineering Profession during the early years of his administration of the Bureau, when he took steps to change the then standard design of naval dry docks from timber to concrete and stone. He had clearly foreseen that wooden dry docks would not be able to meet the requirements of future battleships. Moreover, with their higher costs for maintenance and repairs they were not economical. The criticisms that were directed against him by those interested in the continued construction of wooden dry docks were severe, and attempts were made in various ways to belittle his arguments and to influence his judgment and action, but in vain. His mind had been made up after careful study and con-

sideration, and he remained steadfast in his beliefs. In the face of formidable opposition he convinced Congress of the soundness of his recommendations, which resulted in Congressional authority to change the construction of four dry docks, about to be started, from timber to concrete and stone. That was the last of wooden dry docks for Naval use, and the present standard of concrete and stone construction was thereby established.

Eleven dry docks were constructed under Admiral Endicott's supervision, two of which were steel floating structures. The first of the two floating docks, capable of docking a 16 000-ton vessel, was constructed at Sparrows Point, Baltimore, Md., and towed to New Orleans, La., where it has been in use up to the present time. The famous floating dry dock, *Dewey*, now in service in the Philippine Islands, was the second and the larger of the two. It was designed for docking a 20 000-ton vessel. The *Dewey* far surpassed in size any floating dock that had been constructed up to that time. It embodied some of the features of the New Orleans dock, which was patterned after an English design, but was so far improved and strengthened that the design can be termed an American one. These two docks have self-docking features which permit them to be repaired and painted without utilizing any other methods of docking.

In 1905, Admiral Endicott was appointed by President Roosevelt and confirmed by the Senate as the Navy Member of the Panama Canal Commission. This Commission made its headquarters in Washington, and he continued to perform this duty, in addition to his other work, until 1907, when the Commission was abolished and a new one appointed which made its headquarters and functioned at the Canal Zone in Panama.

Admiral Endicott's efforts were always directed to securing the greatest possible economy in the expenditure of public funds. He advocated and finally obtained the approval of Congress for the consolidation of all power plants at each of the larger Navy Yards into a central station. This resulted in a large saving of Government funds. He originated a design for large floating cranes with a lifting capacity of 100 gross tons and capable of handling guns, turrets, boilers, and other heavy objects.

Following his retirement, Admiral Endicott continued to make his home in Washington, D. C. In 1914, and again in 1917, he was called back to active duty with the Department of Justice, in connection with suits against the Navy Department.

On October 12, 1917, during the World War, he was again ordered to active duty and detailed to the Bureau of Yards and Docks. In addition to various technical duties assigned him, he acted at different times as President of four Naval Examining Boards to examine candidates for appointment to the Corps of Civil Engineers. One of these boards for the appointment of reserve engineers handled and rated the papers of more than 7 000 applicants. In 1918, he was ordered to Norfolk, Va., for temporary duty as a member of a board to report on the extension of the harbor line at the Naval Operating Base at Hampton Roads, Va. He was later appointed a member of the Board

created to recommend those deemed worthy of the award of the Congressional Medals of Honor, the Distinguished Service Medals, and the Navy Crosses, provided for in the Act of Congress approved February 4, 1919. He was relieved from active duty on June 30, 1920. The following is quoted from a letter of commendation given to Admiral Endicott by Secretary of the Navy Daniels for his war service:

"He performed exceptionally meritorious service in a duty of great responsibility, acting in an advisory capacity to the Chief of the Bureau of Yards and Docks, and as a member of various special boards dealing with matters of great importance in connection with the prosecution of the war."

Admiral Endicott was married in Dresden, Ohio, on May 29, 1872, to Elizabeth Adams. He is survived by his widow and seven daughters. His home life was exceedingly happy. He was a devoted husband and father. He was active in church circles, having served as a Vestryman of the Church of the Epiphany (Protestant Episcopal) of Washington. He was a member of the Theta Xi Fraternity; a Trustee of the Protestant Episcopal Church Home of Washington; a life member of the New Jersey Historical Society; a member of the Cosmos Club and Monday Evening Club of Washington; and an Honorary Member of the Washington Society of Civil Engineers.

His health had always been unusually good until within a year and a half of his death and his tall, striking figure was a familiar and always welcome sight on the streets of Washington to his many friends. A heavy cold that he contracted in the latter part of February, 1926, developed into pneumonia and after a few days' illness, he quietly passed away on March 5. Burial was in Arlington National Cemetery on March 8, 1926, where full military honors were accorded him.

In the death of Admiral Endicott the Society has lost an honored member, who cherished a deep interest in its welfare. He was a Naval Officer and Engineer whose patriotism, professional attainments, and high ideals made him one of the outstanding public servants of his day. He always had the courage of his convictions and stood ready to fight if necessary for what he believed to be right. He never temporized where a question of principle was involved, and never took a backward step from the course he determined that he should follow. He has been well called "The Father of the Corps of Civil Engineers." He was an earnest and true friend, wise in counsel, charitable in judgment, kindly, with sympathy for all who appealed to him, and universally beloved by all whose privilege it was to know him.

Admiral Endicott was elected a Member of the American Society of Civil Engineers on April 4, 1877, retaining his connection with it for a period of forty-nine years, lacking one month. He served as a Director from 1901 to 1903; as Vice-President in 1908 and 1909; as President in 1911; and on the Board of Direction, as Past-President, from 1912 to 1916.

HENRY FLEETWOOD ALBRIGHT, M. Am. Soc. C. E.*

DIED MAY 11, 1926.

Henry Fleetwood Albright was born at Lancaster, Pa., on October 5, 1868. He received his education in the public schools of Philadelphia, Pa., and shortly after his graduation went to work as an office boy with the Union Pacific Railway Company in that city.

In 1892, while employed as a Sales Engineer for the Thompson-Houston Company, he attracted the attention of the Western Electric Company when he secured a contract from three of its best salesmen for an electric lighting plant for Wheeling, W. Va. The latter Company appreciating the efficiency of this transaction offered Mr. Albright the position of Sales Engineer which he accepted.

In 1894, he was transferred to the Construction Department of the Western Electric Company, in New York, N. Y., where as Supervising Electrical Engineer he designed and supervised the installation of electric power and lighting systems. His work was done in such an efficient manner that in 1897 he was promoted to the position of Factory Engineer, in charge of the design and supervision of the installation of electrical and mechanical building equipment, such as electric current generation, distribution for power and lighting, heating, sprinklers, plumbing, and elevators. He also supervised the installation of manufacturing equipment, such as machine tools, benches, and storeroom equipment in the New York Shop of the Company.

From 1898 to 1899, he served as Assistant Shop Superintendent and was in responsible charge of preparing layouts of manufacturing operations, including the routing of materials through the shop, as well as layouts of manufacturing equipment and store rooms.

From 1899 to 1908, Mr. Albright held the position of Shop Superintendent and his talent for systematic organization soon made the New York factory a model of efficiency. Realizing the probable rapid growth of the Company, he began to develop his organization along functional lines, grouping together, under responsible heads, departments that performed the same general function.

In 1908, Mr. Albright was made General Superintendent in charge of all manufacturing operations of the Western Electric Company in the United States. It was at this time that he moved to Chicago, Ill., where he made the Hawthorne Works, then being erected, his headquarters, and began the process of concentration of the Company's principal manufacturing activities at that plant.

The Hawthorne Works which is one of the largest and most complete manufacturing plants in the United States is the greatest testimonial to Mr. Albright's business ability. The first buildings at this plant were erected in 1905 and he supervised the layout for the Telephone Apparatus Shops and also an ultimate layout for the entire plant which has been consistently followed as various buildings have been added. During the period from 1908 to

* Memoir compiled from information on file at the Headquarters of the Society.

1923, the total floor space used at the Hawthorne Works for manufacturing was increased from 300 000 to 3 000 000 sq. ft. and the number of employees from 2 500 to 30 000. In the buildings of this plant, Mr. Albright installed the latest and best ideas in methods, equipment, and arrangement of machinery, thereby making the Works as a whole a model in factory building.

In addition to his duties for the Western Electric Company in the United States, Mr. Albright was also in responsible charge of the selection of sites and the design of buildings and manufacturing layouts for the ultimate development of properties of its foreign and affiliated Companies at Antwerp, Belgium, London, England, Paris, France, and Tokyo, Japan.

In 1923, Mr. Albright was made Vice-President and Director of the Western Electric Company, Incorporated, which position he held until his death. In this position he retained his duties as General Superintendent in charge of the manufacturing operations of the Company and in addition he was placed in charge of the selection of a site for a new manufacturing plant at Kearny, N. J., the design of the buildings, and the development of manufacturing equipment layouts for that plant.

He died on May 11, 1926, at the Memorial Hospital in New York, N. Y., after a long illness. His body was taken to his home in Oak Park, Ill., and buried in Glen Oak Cemetery. On the day of his funeral the Hawthorne Works were closed as a mark of respect.

On December 26, 1892, Mr. Albright was married to Laura Heston, of Philadelphia, Pa., who, with a son, Henry F. Albright, Jr., and a daughter, Laura, survives him.

Relative to his ability as an engineer and executive and to his character as a man, Charles G. Du Bois, Chairman of the Board of Directors of the Western Electric Company, and a close business associate of Mr. Albright, has written the following appreciation:

"Mr. Albright was a man of unusual and marked abilities. He did his work carefully and completely. He envisioned great projects but he never overlooked or despised the details. As his responsibilities grew he trained others to take care of the lesser things and he trusted them. He was vigilant in the Company's interest on the daily work, but his mind constantly turned toward its future operations and the preparation to be made for them. He is gone but of no man can it be more truly said that his work lives after him.

"And yet it is not the superb and enduring work he left behind which has filled my thoughts of him in these past few weeks. It is his personality—the man himself apart from the work he did.

"Nowhere did he reveal himself to me so fully as in our travels together. His capacity for absorbing information on any subject, the wide range of his intellectual curiosity, and his remarkable memory made him always a most interesting traveling companion. And the mere being in motion on ship or train seemed to stimulate his memory of other scenes and incidents or stories so that he entertained his fellow-travelers much more I think than he supposed. For although completely fearless, he was naturally shy and self-effacing, usually concealing his inner feelings and covering with a mantle of scientific detachment and coolness a deeply emotional nature. He was a true scientist, unswerving in his search for facts and their meanings, but a discriminating taste, a spirit of artistry, and a love of beauty in every form colored and

animated his outlook. This combination of qualities would have made him a rare personality but he added to them an affection for and appreciation of his friends and associates which inspired in them a devotion to him such as few men ever achieve."

He was a member of the Institute of Electrical Engineers, of London, England; a member of the American Society of Mechanical Engineers; and a Fellow of the American Institute of Electrical Engineers. He was also a member of the Union League Club, the Lotus Club, and the Oak Park Country Club of Chicago. He was a member of the Presbyterian Church.

Mr. Albright was elected a Member of the American Society of Civil Engineers on December 14, 1925.

MILES CARLISLE BLAND, M. Am. Soc. C. E.*

DIED AUGUST 14, 1926.

Miles Carlisle Bland was born in Philadelphia, Pa., on July 31, 1875. His parents were John Carlisle and Emma Virginia Bland. He was graduated from Colorado College in 1895 and matriculated at the University of Pennsylvania with the intention of taking the Civil Engineering Course. An offer of a position in the Maintenance of Way Department of the Pennsylvania Lines West of Pittsburgh changed his plans, however, and, in September, 1895, he began his professional career as an Assistant with the Engineering Corps. In the next six years he secured the broad training in general railroad engineering that such a position affords.

In 1901 Mr. Bland went with the American Bridge Company as Assistant Engineer, later becoming Contracting Manager of the Cleveland, Ohio, Office. From 1905 until 1907 he was engaged on the construction of beet sugar factories in Michigan and Colorado, representing the Dyer Company, of Cleveland. In 1908 he returned to the steel fabricating field as Contracting Engineer in Pittsburgh, Pa., for the Pittsburgh Steel Construction Company, with subsequent engagements with the Massillon Bridge Company and the Canton Bridge Company. He returned to Cleveland in 1913 as Resident Engineer on the construction of the foundation for the Detroit Superior High Level Bridge, built by Cuyahoga County over the Cuyahoga River Valley.

In 1916 Mr. Bland went with the Bethlehem Steel Bridge Corporation as Engineer in the Erection Department. The complete and varied erection experience which he acquired while on this work was reflected in his paper on "Investigation of Stresses in Derricks",† presented before the Society in 1921, and his "Hand Book of Steel Erection" (1923).

In March, 1923, he joined the Engineering Staff of the Delaware River Bridge Joint Commission as Assistant Engineer in the Design Division and subsequently was transferred to field work as Resident Engineer on the erection of the main cables of the bridge over the Delaware River, between Philadelphia, Pa., and Camden, N. J., until August, 1925. From that date until his death on August 14, 1926, he was with the City of Philadelphia as Assistant Engineer of Bridges.

Miles Bland brought a wide and varied experience to his later professional engagements. His friends knew him as an unselfish gentleman, unsparing of himself in the application of his high standards of performance to the task in hand.

* Memoir prepared by Clement E. Chase and Frank M. Masters, Members, Am. Soc. C. E.
† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 258.

He held a commission as Captain in the Ordnance Reserve and was a Member of the Society of Military Engineers; the National Rifle Association of America; the American Society of Mechanical Engineers; and the American Association for the Advancement of Science.

He is survived by his widow and infant son, as well as a son by his first marriage.

Mr. Bland was elected an Associate Member of the American Society of Civil Engineers on September 3, 1902, and a Member on April 21, 1920.

JOSÉ ANTONIO CANALS, M. Am. Soc. C. E.*

DIED FEBRUARY 28, 1926.

José Antonio Canals was born in Guanabacoa, Cuba, on February 20, 1859. His parents removed to Porto Rico shortly after his birth. He attended the College Seminario of San Juan, Porto Rico, conducted by the Jesuit Fathers, and received his degree of Bachelor of Arts in 1874. Soon afterward he left for Belgium where he attended the School of Civil Engineering of the University of Ghent, graduating therefrom in 1883, and returning to Porto Rico the same year.

From 1883 to 1885 Mr. Canals was in charge of the survey and design of an irrigation system for the District of Guayama, Porto Rico, of which the estimated cost was $500 000 and, in 1886, he designed improvements and the extension of the water-works at Guayama.

From 1886 to 1888, still under the Spanish Government, he served as City Architect of San Juan, leaving this position to accept one with "La Société d'Entreprises et de Constructions des Colonies Espagnoles", General Contractors of the Porto Rican Railroad Company for the construction of 194 km. of road, with which he remained until 1895, having been engaged as follows: From January, 1889, to 1890, he served as Assistant Engineer; from 1890 to 1892, as First Assistant to the Chief Designing Engineer; from 1892 to 1893, as Chief Designing Engineer; and from 1893 to 1895, as Representative of the Company in connection with the liquidation of its contract.

Mr. Canals then accepted a position with the Porto Rican Railroad Company, acting as Chief Engineer of the rolling stock, from 1896 to 1898, at which time he was transferred to the Maintenance of Way Department. During 1898, after the Spanish-American War, the Americans took possession of the Island of Porto Rico, and Major General Brooks, knowing the interest and skill of Mr. Canals, appointed him Engineer of Public Works of Porto Rico, in charge of surveys and the construction of roads. He held this position until 1900, at which time he returned to the Porto Rican Railroad Company as Chief Engineer on the repair and reconstruction of bridges.

In 1902, Mr. Canals was appointed City Engineer of San Juan, by Governor Hunt, and as such superintended the repair of the water-works and studied conditions to prevent the waste of the water supply. Under great difficulty and in the face of much bitter opposition he succeeded in introducing water meters for the conservation of the water supply and thus curtailed the enormous expense which had previously drained the Municipal Treasury. This measure was passed only after much heated discussion, attacks by newspapers, and denouncement by the public, but his integrity of character never wavered and he installed the meters, the consumption decreasing from 235 liters per capita per day in 1902 to 80 liters in 1905. At the same time he was engaged in completing the sewerage system and street paving.

* Memoir prepared by J. M. Canals, Jun. Am. Soc. C. E.

From 1905 to 1919, Mr. Canals had an office as Consulting, Contracting, and Designing Engineer, and was in charge of the construction of a number of large buildings. He also saved the Loiza Sugar Company and the Canóvanas Sugar Company about $1 500 000 by preventing the replacement of their 75-cm. railway by a 1-m. railway. This was an important piece of work, for a bill was before the House of Representatives demanding the removal of the 75-cm. tracks.

In 1919, his services were again required by the municipality of San Juan on account of an insufficient water supply due to an extremely dry season. Mr. Canals' strength of character against great opposition and his unwavering purpose in giving his best to the people in 1902 were remembered and fully appreciated, and during this crisis he was called to advise and accepted the position of Superintendent of Public Works.

In 1920, he left the Municipal Service and re-opened his office as Consulting Engineer, but, in 1923, he was recalled again by the City Government to consult with other prominent engineers on the revision of the project of new water-works for San Juan. Subsequently, he was appointed Chief Engineer in the Office of City Improvements, taking charge at the end of 1923 of the Office of Director of Public Works.

In March, 1924, the party that was in power had to yield to the opposition. The latter, keenly aware of Mr. Canals' sterling worth, felt that it could not do without his assistance and offered him the position of Consulting Engineer of the City of San Juan, which position he held until his death. During this last period of his life he was engaged zealously in the location, construction and re-adjustment of the new water-works of San Juan.

In addition to his various positions in the engineering field, Mr. Canals occupied a number of important offices in the Insular and Municipal Government. On April 14, 1904, he was appointed by Acting Governor Charles Hartzell of Porto Rico, a member of the Superior Board of Health, and on September 21, he was appointed a member of the Municipal Council by Governor Arthur Yager. He was also made a member of the Workmen's Relief Commission on May 27, 1918. On December 18, 1922, he was appointed by Acting Governor J. B. Huyke as President of the Mediation and Conciliation Commission.

Outside his official duties, Mr. Canals' interest was especially in his home, where he died on February 28, 1926. He is survived by his widow, a son, and three daughters.

In 1911, he was President of the Porto Rican Society of Civil Engineers, having been at the time of his death an Honorary Member of that Society.

Mr. Canals was elected a Member of the American Society of Civil Engineers on May 3, 1905.

PHILLIP WILLIAM CHAMBERLAIN, M. Am. Soc. C. E.*

DIED DECEMBER 3, 1926.

Phillip William Chamberlain, the son of Henry William and Concepcion Murillo y Martoviell Chamberlain, was born in Essex, N. H., on June 14, 1853. One of his uncles was an Admiral in the British Navy and another an Admiral in the Spanish Navy. His father was a Mining Engineer and a Lawyer, and out of sympathy with the Confederacy, he left Essex in 1861 and moved to Cuba with his family. Here his son attended the Grammar School and was graduated in 1868.

Mr. Chamberlain began the practice of his profession as Chainman and Rodman for the Chief Engineer of the Consolidated Copper Mines in Cuba, on whose advice he entered St. Iago College, at Santiago, Cuba, from which he received his degree of Bachelor of Arts, in 1872. He then entered the University of Havana for a course in engineering, including sugar making, its chemistry and machinery, and also international law. He received his degree of Master of Arts in 1875.

From Cuba, Mr. Chamberlain went to Costa Rica, where he remained for several years working on the construction of the Costa Rica Railroad with Mr. Minor C. Keith. Mr. Chamberlain and Mr. Keith were friends whose acquaintance lasted many years.

From Costa Rica, Mr. Chamberlain went to Nicaragua, where he surveyed a railroad from San Juan del Sur to Lake Nicaragua. He was then appointed Superintendent of the Eastern Division of the Nicaraguan National Railroad. In 1890 he accepted the position of Chief Engineer of the projected railroad from Leon to Matagalpa (150 miles), having made the surveys and maps for this work.

He designed and built the Monotombo Wharf at Lake Managua in 1892, and in the same year was appointed General Manager of the Nicaraguan National Railroad, which position he resigned to accept that of Engineer with the Guatemala Central Railroad, Guatemala. By contract with the Guatemala Government, he then surveyed and built the Iztapa Railroad.

In 1899, he returned to Nicaragua and was appointed Consulting Engineer for the Department of Public Works. While in this position he surveyed and built 17 miles of railroad at Greytown.

Mr. Chamberlain retired from active engineering work in 1905 and became a farmer. He owned a banana plantation, sawmill, and an electric light plant. A blight destroyed the banana fields before the first crop This ruined him financially but he sold everything, paying his creditors in full.

In 1914, Mr. Chamberlain surveyed plantation tramways for farmers, and made a preliminary survey for a railroad from Alajuela to Gracia, Costa Rica. The termination of the World War found him very ill both in body and spirit, and ready to return to the United States, but he could not procure transportation.

* Memoir prepared by Mrs. Eleanora W. Chamberlain, Baltimore, Md.

When he returned to the United States in 1919 he found employment with the Bureau of Highways at Baltimore, Md., as Associate Engineer. Subsequently, he went to the Bureau of Sewers, where he remained two years, retiring on account of ill health.

From 1919 to 1926, Mr. Chamberlain maintained his residence in Baltimore, where his death occurred on December 3, 1926, after two years of patient suffering.

In 1905 he was married to Eleanora Wagner, of Baltimore. His wife survives him.

Mr. Chamberlain was a Spanish scholar. He wrote a book on "The Volcanoes of Nicaragua," which was published in 1902, and contributed many articles in Spanish and English on engineering, history, astronomy, international law, and religion, to periodicals in Central America and the United States. He spoke and wrote five languages.

He was one of those men of whom the world sees too few, and all who came in contact with him know his real worth.

Mr. Chamberlain was elected a Member of the American Society of Civil Engineers on October 2, 1895.

ANDREW OSWALD CUNNINGHAM, M. Am. Soc. C. E.*

DIED MAY 9, 1926.

Andrew Oswald Cunningham was born in Rangoon, British Burmah, on July 8, 1866, the son of Maj. Gen. Percy S. Cunningham, of the British Army, and Anna Sarah (Stroud) Cunningham. His father held many commissions of trust in the Government Service while in Burmah.

Mr. Cunningham attended South-Eastern College, at St. Lawrence, England, from 1879 to 1883, when he came to the United States. He took up railroad work and was, successively, Rodman and Leveler with the Northern Pacific Railroad Company from 1886 to 1888. He then engaged in land surveying in North Dakota for two years.

In 1890, at the age of 24, he entered the University of Minnesota and was graduated in 1894, with the degree of Bachelor in Civil Engineering. His classmates recall with affection his likeable personality and his sense of humor. Having attended school in England, Mr. Cunningham was an expert at Rugby football and astonished coaches and students alike with his proficiency. He took his university work more seriously than the average student because of his more mature years, and found less time for the lighter phases of student life, although he was generally popular. While pursuing his engineering studies he was engaged in constructing insurance maps of the City of Minneapolis.

Following his graduation Mr. Cunningham became associated with the Gillette-Herzog Manufacturing Company, of Minneapolis, in the capacity of Draftsman and, in 1895 and 1896, was Assistant Engineer of the Company. From 1896 to 1898 he was Southern Agent for the Schultz Bridge and Iron Company of Pittsburgh, Pa. Succeeding engineering engagements were with the Pittsburgh Reduction Company, in 1898 and 1899; general construction and engineering, in 1899 and 1900; and in charge of estimates, designs, and bids for the American Bridge Company, Cleveland, Ohio, from 1900 to 1902.

In 1902 Mr. Cunningham removed to St. Louis, Mo., where he became Bridge Engineer of the Wabash Railroad Company. He was promoted to the position of Chief Engineer in 1905 and continued in that capacity until 1923 when he was made Consulting Engineer. He resigned from this position in 1924 to engage in consulting practice. He was Consulting Engineer of the Terminal Railroad Association of St. Louis from 1924 until his death, which occurred at his home in University City, Mo., on May 9, 1926, at the height of his career.

Mr. Cunningham was a man of active mentality, great breadth of vision, and marked originality. His resourcefulness in rehabilitating and reinforcing obsolete bridges, increasing their capacity to carry modern loading, by unique methods and original designs, thus avoiding the expense of replacement, won for him much high commendation. He perfected many inventions, and many

* Memoir prepared by the following Committee: O. F. Harting, Assoc. M. Am. Soc. C. E., *Chairman*, R. H. Howard, and W. E. Rolfe, Members, Am. Soc. C. E.

mechanical devices were the result of his thought and study. He was the first to advocate the use of reinforced concrete in the construction of bridge floors, and the Cunningham mattress for river-bank protection has been widely used.

He was much interested in civic affairs and served a two-year term as Alderman of University City, Mo., beginning in April, 1914. He was elected Mayor of University City in April, 1925, but died before the expiration of his term of office. While serving as Alderman he prepared and negotiated a contract securing to University City one of the best and cheapest street-lighting systems in any city of the United States. During his tenure as Mayor many municipal improvements were carried out under his direction, and his experience and judgment in engineering and construction work were invaluable to the Engineering Department of the City.

The death of Mr. Cunningham is a distinct loss to all with whom he came in contact; his friendship and counsel will be missed by all his former associates and he leaves a void in the ranks of the Engineering Profession that will be difficult to fill.

Mr. Cunningham was a member of the American Railway Engineering Association and the Engineers' Club of St. Louis, in both of which he took an active part, having served as Chairman and member of a number of their important committees.

He was married on July 11, 1903, to Mrs. Georgia Townsend Quinn, of St. Louis. In his home he was a perfect host. His charity was boundless, without regard to sect or creed. It is fitting that his memorial be concluded in the words of his devoted wife and helpmate:

"With him honor came first at all times, then love for his family and friends; he cared only for a peaceful and honorable life, trying to bring happiness where he could."

Mr. Cunningham was elected a Member of the American Society of Civil Engineers on February 5, 1902.

AUGUSTINE LEE DABNEY, M. Am. Soc. C. E.*

DIED AUGUST 23, 1926.

Augustine Lee Dabney was born on November 17, 1870, at Vicksburg, Miss. During the summer of 1886 he served as Rodman on levee construction work and on railroad preliminary surveys, and in the summers of 1887 and 1888 he worked on levee construction as Instrumentman and on railroad location, resigning this position to prepare for college.

He attended Washington and Lee University, at Lexington, Va., during the session of 1888-89, and the following summer became First Assistant Engineer on levee construction, Topographer, and, later, Precise Recorder in the United States Survey of the Red River, Louisiana. In 1890 he served as Resident Engineer on the Yazoo-Mississippi Delta Levee District in charge of constructing five miles of new and enlarging four miles of existing levees.

Mr. Dabney returned to Washington and Lee University for the sessions of 1890-91 and 1891-92 and was graduated in 1892 with the degree of Civil Engineer and high rank in a large class.

From June, 1892, until May, 1896, he was Assistant United States Engineer in charge of all Federal work in the Lower White River Levee District, directing the prosecution of work costing about $400 000 and having full charge of high-water and flood protection. He made locations and estimates for new levee lines and passed on the completed work which he did with such success that he was promoted to be Assistant Chief Engineer of the Yazoo-Mississippi Delta District, in which capacity he served from 1897 to 1902. Many of the wonderful works planned and carried out there stand as a monument to him.

In February, 1902, Mr. Dabney left the Government work, engaging in private practice, but after successfully carrying out several important projects of sewage disposal, drainage, and reservoir engineering, he was again called to serve the Government, this time as Assistant Chief Engineer of the Tallahatchie Drainage District, comprising 1 150 000 acres of the Yazoo-Mississippi Delta. After giving two more years to this work, he again entered the more profitable field of private practice as a specialist and acknowledged authority on drainage, flood protection, sewage disposal, and related engineering work.

On October 18, 1905, while engaged in prosecuting the work of the Tallahatchie Drainage District, Mr. Dabney was married to Margaret Munson, of Chicago, Ill., who had been sent as School Mentor to Columbus, Miss., by the Chicago Normal School of which she was a graduate.

In 1917 he entered the Military Training Camp at Fort Oglethorpe, Ga., and was commissioned a Major of Engineers, United States Reserve Corps, in which capacity he did signal service in designing cantonment sewers and supervising such construction.

* Memoir prepared by Arthur J. Dyer, M. Am. Soc. C. E.

At the close of 1917 Major Dabney investigated and reported flood protection for the Chattanooga, Tenn., District—a project variously estimated at between $3 500 000 and $6 500 000 and, in 1918 and 1919, he held the position of Engineer in Charge of Water-Works, Sewerage, and Drainage, at Muscle Shoals, Alabama, the big nitrate plant. After the World War he returned to private practice at Memphis, Tenn., in which he was successfully engaged until his death.

The circumstances of Major Dabney's death were tragic. He and Mrs. Dabney both gave their lives in an heroic effort to rescue their little adopted daughter, Sarah Lee, who was also burned to death when their home was destroyed by fire.

During his long and useful career Major Dabney was the author of many technical publications well and favorably known to engineers and others interested in the subjects in which he specialized. These are generally recognized as valuable and authoritative contributions to this important branch of the profession among others being a noteworthy treatise on the relation between precipitation, run-off, and discharge.

Major Dabney was a member of the Illinois Society of Engineers and the National Geographic Society, and of the Engineers' Club, the City Club, and Chamber of Commerce of Memphis, Tenn., in which city he made his home and practiced his profession during the latter years of his life.

He was a gentleman of the highest order of intelligence and of the strictest integrity; a man of fine scientific attainment combined with not only highly specialized, but well balanced practical knowledge and experience, which, with unusually accurate and impartial judgment, pre-eminently fitted him for membership on the Tennessee State Board of Architectural and Engineering Examiners, on which Board he served conspicuously as Chairman. In his death the Engineering Profession has lost a useful member and the community in which he lived an excellent and eminent citizen.

Major Dabney was elected a Member of the American Society of Civil Engineers on October 2, 1901.

JAMES WORK DEEN, M. Am. Soc. C. E.*

DIED JULY 27, 1926.

James Work Deen was born on August 4, 1852, in West Waterford, Pa., the son of James and Rosanna (Work) Deen. He was a direct descendant of Richard Stone, Private in Captain Henderson's Company, Colonel Harmer's Regiment, First Battalion, Pennsylvania Continental Line. His education was acquired in the public schools of Pennsylvania and in the State Normal School.

Mr. Deen entered railway service as Rodman and Levelman on location of the Fulton and Franklin Railroad in 1877. The following year he was Instrumentman on the construction of the Selinsgrove and North Branch Railroad, and from April, 1879, to April, 1881, he was, successively, Rodman and Instrumentman, on Maintenance of Way, Middle Division of the Pennsylvania Railroad.

In 1881 he left the service of the Pennsylvania Company and joined the Engineering Corps of the Denver and Rio Grande Railway Company, with which Company he was thereafter continuously employed until the time of his retirement in 1923, a period of 42 years. One of Mr. Deen's earliest works of responsibility, during practically a lifetime of exacting railway engineering in the mountains, consisted of rebuilding 20 miles of line in Grape Creek Canyon, destroyed by the flood of 1881. His next notable position of trust lay in the rectification of alignment on Marshall Pass and in the Black Canyon of the Upper Gunnison River.

Afterward, in 1886, his remarkable talents for location were recognized in his appointment as Locating Engineer, and a year later he was placed in charge of construction on the Eagle River Extension, now a part of the main line between Denver, Colo., and Salt Lake City, Utah. In the winter of 1887-88, Mr. Deen was in charge of widening the line from Pueblo to Trinidad, Colo., to standard gauge, and early in 1888 he was appointed Division Engineer of the Second and Third Divisions of the Denver and Rio Grande Railway, the former including the lines on the Marshall Pass route between Salida and Grand Junction, Colo., and the latter the lines on the Tennessee Pass route between Salida and Aspen, Colo.

It was during Mr. Deen's twenty-five years of service in this capacity that his most effective and lasting work as an engineer was accomplished. On the location in 1889 and in charge of the construction in 1890 of the standard gauge line over Tennessee Pass and through the Eagle and Colorado River Canyons, his ingenuity and engineering ability were brought into full play, and the results accomplished still bear witness to the remarkable ability of the man.

Mr. Deen never left the mountains in which he had spent the greater part of his life and to which he became firmly attached. Upon his retirement

* Memoir prepared by Arthur O. Ridgway, M. Am. Soc. C. E.

from active service he continued to reside at Salida, a respected citizen of the community.

In June, 1891, Mr. Deen was married to Martha J. Snyder, of Mifflintown, Pa., who, with two of his sisters, Mrs. Lillie D. Mosgrove, of Salida, and Mrs. C. P. Lantz, of Washington, D. C., survives him.

For thirteen years he served as a member of the Salida City Council, and always took more than an ordinary interest in civic, State, and National affairs. As a mark of honor, all public and business institutions of Salida were closed during the funeral ceremonies. No better tribute can be paid to his memory than that of the *Salida Mail* in its issue of July 30, 1926:

"The average citizen is an asset to a community while he lives in it, and a loss to it when he moves away or answers the final call. But it is given to some men to be of special value to the town and State in which they live. J. W. Deen was one of these. He was one of Salida's best citizens and one of the builders of the State."

He was a member of the Colorado Society of Engineers, Sons of the American Revolution, the Salida Chamber of Commerce, and an Honorary Member of the Salida Lions Club.

Mr. Deen was elected a Member of the American Society of Civil Engineers on January 6, 1892, and became a Life Member in 1925.

HARVEY JACOB DRESSLER, M. Am. Soc. C. E.*

DIED DECEMBER 9, 1926.

Harvey Jacob Dressler was born at Dayton, Ohio, on December 31, 1886. His early education was received in the public grade schools and High School of Dayton. Later, he entered Ohio State University at Columbus, from which he was graduated in 1909, with the degree of Civil Engineer.

From 1910 to 1912, Mr. Dressler was employed by the City of Dayton as Assistant City Engineer. The partnership of Cellarius and Dressler, Civil and Consulting Engineers, was formed in June, 1913, by Frederick J. Cellarius, M. Am. Soc. C. E., and Mr. Dressler, with headquarters in Dayton.

During the World War Mr. Dressler served with the 308th Engineers in France from June, 1917, to July, 1918, and after the signing of the Armistice, was with the Army of Occupation at Coblenz, Germany, until early in 1919.

He died on December 9, 1926, at St. Elizabeth's Hospital, in Dayton, while submitting to an operation for the removal of his tonsils.

Mr. Dressler is survived by his widow, Mrs. Florence Dressler, and two daughters, Mary Eleanor and Dorothy Ann.

He was a member of the Engineers Club of Dayton, the American Legion, St. John's Lodge of Masons, and the Scottish Rite. He was also an active member of Hope Evangelical Lutheran Church, in Dayton, where he led a Young Men's Bible Class.

Mr. Dressler was elected a Member of the American Society of Civil Engineers on March 15, 1926.

* Memoir prepared by C. S. Bennett, Assoc. M. Am. Soc. C. E.

ALLEN DOUGLAS DUCK, M. Am. Soc. C. E.*

DIED JUNE 3, 1926.

Allen Douglas Duck, the son of Joseph George and Minnie (Fink) Duck, was born at Beaver Falls, Pa., on August 2, 1884. In his infancy the family moved into Ohio, locating, for a short period, at Salem. They then removed to Wellsville, in which town Mr. Duck received his public school education.

On graduation from High School he secured a place with the City Engineer of Wellsville as a Rodman on work being done there. Rapidly advancing from this position, he became Assistant City Engineer, which place he ably filled until late in 1904.

In January, 1905, he entered the service of the United States War Department, where, in teaching mathematics, in surveying, in laying of submarine mines and cables, in handling high explosives, etc., he laid the foundation on which later was built his enviable record as an engineer. After almost six years of this service he resigned his position with the War Department to take up general engineering practice with Prout and Wells, of Aurora, Ill.

As he was strong of body, and a keen lover of out-of-door life at its best, the opportunity for satisfying such a nature seemed to lie in Government service; so he returned to that service as a Recorder on Topography with the United States Geological Survey. After covering many of the States in this work—making an enviable record as an Assistant Topographer—Mr. Duck went to Greenville, Tex., in December, 1912, to accept the position of Assistant City Engineer. His valuable experience, together with his ability and capacity for hard work, won for him after a short time the place of City Engineer.

Devoted to his profession, and with the welfare of his adopted home grounded deeply within him, he began a task of construction and re-construction which grew with the years. No improvement of the city was contemplated without being first submitted to him for approval. To look after the need of a fast-growing city called for just such a man as Mr. Duck. Untiring, happy in the accomplishment of some difficult task, he spent whole days and nights in the solution of his more difficult problems.

Completing an extensive improvement program in Greenville in 1918, he tendered his resignation as City Engineer and entered private practice, acting as City Engineer in several Texas municipalities in the design of sewerage and water-works systems, or the paving of their streets.

In 1919, he was called on by the County Commissioners of Hunt County, Texas, of which Greenville is the county seat, to accept the position of County Engineer to supervise the design of seven cardinal roads running through the County. He accepted this appointment and carried the work through until the project was almost completed, when with reluctance

* Memoir prepared by Malcolm Ramsay, Assoc. M. Am. Soc. C. E.

the Commissioners accepted his resignation, and he again devoted his time to the municipal problems of Greenville.

The constant drain on his facilities at this time undermined his health, and he was compelled to stop work and seek rest for a short period. His indomitable will soon seemed to gain the upper hand in the struggle, and he again plunged into his work, seemingly as robust as ever.

In 1925, Mr. Duck joined the ranks of contractors, as a Contracting Engineer, accepting a contract under the City of Greenville to construct all concrete features of its new reservoir. Completing this work, he accepted, in the spring of 1926, a contract to pipe this water to the city in large mains. All construction machinery had been purchased and was on the ground, all material had been ordered, and everything made ready to commence the work when Mr. Duck was suddenly stricken at his home and passed away before medical aid could reach him. Thorough in all undertakings, he was one who believed that, once a task is taken up, it should be worked through to a conclusion, and, knowing this, his wife stepped into his place and brought the work to a successful conclusion.

To have known Mr. Duck was a great pleasure—his personality, like the rays of the sun, lighted the dark places and warmed the cold. To have worked beside him was an honor. Ever alert to his task, he was looked on by his business associates as the personification of honesty and integrity. His father died during Mr. Duck's infancy, necessitating the curtailment of his technical education. He had the will to do; hence, by home study and with a clear mind as an asset, he surmounted this obstacle to his success. At the end of a far too short life, he was abreast of the foremost engineers of Texas.

A self-made man, Mr. Duck generously assisted those young people who were forced, through like circumstances, to travel the road over which he had so recently come. Generously, and ever quietly, he helped to smooth life's pathway for many of them. The esteem in which he was held by his fellow townsmen was manifested during his funeral services, when all municipal activities ceased for several minutes as a mark of respect to the man who had meant so much in the life of his town.

He was united in marriage on January 27, 1914, to Mildred McSpadden, of Greenville. To this union were born two children, Douglas and Mildred Frances. His home life was ideal, and his whole life was centered on those who formed the little family circle.

Mr. Duck was a true Christian and a member of the Christian Church. He belonged to all branches of the Masonic fraternity.

During the last few years of his life he became interested in stamps and had acquired a wonderful collection. He was a member of the American Philatelic Society. His community will long remember him for his engineering ability—his friends for his manhood.

Mr. Duck was elected an Associate Member of the American Society of Civil Engineers on September 12, 1916, and a Member on October 8, 1918.

GEORGE STOVALL EDMONDSTONE, M. Am. Soc. C. E.*

DIED MARCH 23, 1926.

In the passing of George Stovall Edmondstone on March 23, 1926, the Engineering Profession lost a member of great ability, and the community, a gentleman of the old school. To him honor was of first importance, and loyalty to principles and friends a predominant quality. To many younger engineers he was an inspiring influence in fixing high ideals of professional and personal integrity.

George Stovall Edmondstone, always known as "The Captain" because of his association with Army work as Civilian Engineer, was born in Augusta, Ga., on January 22, 1864. The Edmondstone family is of Scotch descent; the first member to become an American, settled in Charleston, S. C., in the Eighteenth Century. George Stovall Edmondstone's father, Henry Edmondstone, was a cotton broker in Augusta, whose business was ruined by Sherman's famous march to the sea. He afterward for many years was Cashier of the Georgia Railway.

George Stovall Edmondstone's professional education was obtained in part at the Carolina Military Institute at Charlotte, N. C., from which he was graduated in 1882. After a year spent in railroad construction work in his native State he continued his engineering training under the late James B. Francis, Past-President, Am. Soc. C. E. Following his hydraulic work under Mr. Francis, he served as Assistant City Engineer of Augusta for several months, being engaged in sewer and hydraulic work. The years from 1886 to 1890 were spent as Construction and Designing Engineer for a New York firm financing and building water supply systems for various cities, including Greencastle, Md., and Defiance, Ohio. He served as City Engineer of Defiance from 1890 to 1892.

Leaving municipal work in 1892 he went to Wyoming, where for five years he was in private practice, engaged chiefly in irrigation and water power projects. In 1900 he returned to municipal work and for four years held the position of Bridge Engineer for the City of St. Paul, Minn. The bridges designed and constructed by Mr. Edmondstone included those on Western Avenue, Arcade Street, Rice Street, and Grand Avenue. From 1904 to 1907, he was Construction Engineer for the United States Government at Forts Keough and William Henry Harrison, in Montana. This again was chiefly hydraulic work.

Three years, 1907 to 1910, were spent in private practice with headquarters at Helena, Mont. His work at this time was varied, but was chiefly hydroelectric and structural. During this period he was Construction Engineer for the Missouri River Power Company, reporting on the failure of the Houser Lake Dam. Water-right determinations for irrigation companies and irrigation construction occupied part of his time during this period.

* Memoir prepared by Philip H. Dater, M. Am. Soc. C. E.

In 1911 Mr. Edmondstone undertook the investigation, plans, and estimates of the Clear Lake (McKenzie River), Oregon, water power project of the Oregon Electric Company. Three years were spent on this work. The project is an outstanding testimonial of his professional ability. He transformed a dubious project into one of certainty, materially increasing the power capacities, and greatly reducing the estimated cost. This accomplishment is typical of much of his other work.

From 1914 to 1917, he was in charge of the design and construction of important railroad grade separations in Portland, Ore., notably the Sullivan's Gulch grade separation of the Oregon-Washington Railroad and Navigation Company. The following two years were spent as Construction Engineer in charge of building the Portland Municipal Grain Elevator and Terminal No. 4 at St. Johns, Ore. From January, 1921, to July, 1922, he served as Engineer for the Portland School Board, being in charge of school properties.

In 1922, Mr. Edmondstone engaged in private engineering practice and in contract work in which he continued until his death. The fuel conveyor plant of the Long-Bell Lumber Company, at Longview, Wash., is one of the accomplishments of this period of a busy and useful life.

In 1899 he was married to Emilie von Bergen, at Iowa City, Iowa. Mrs. Edmondstone, a son, George Stovall, and three daughters, Claire, Annie, and Elizabeth, survive him.

No outline of Mr. Edmondstone's life would be complete without mention of his active interest in the Masonic fraternity. Few members of the order were better versed in its history and traditions, nor more loyal in carrying its teachings into practice.

Mr. Edmondstone was elected a Member of the American Society of Civil Engineers on November 27, 1917.

CHARLES GLEASON ELLIOTT, M. Am. Soc. C. E.*

DIED SEPTEMBER 14, 1926.

Charles Gleason Elliott was born in Lowell, Ill., on June 8, 1850, the son of John B. and Elizabeth (Searles) Elliott. He was educated at Illinois Wesleyan University, Oberlin College, and the University of Illinois, receiving the degree of Bachelor of Science from the latter institution in 1877, and that of Civil Engineer in 1893.

Following his graduation Mr. Elliott engaged in general engineering practice, principally in drainage engineering in the North Central States, and in geologic investigations in the oil fields of the West.

In 1884, he was appointed Sanitary Engineer by the Superintendent of the Hospital for the Insane at Indianapolis, Ind., to investigate and report on a plan for the drainage and sewerage of that institution. In 1886, he served as Chief Engineer for the Red River Valley Drainage Commission in charge of the surveys and design for the drainage of 809 000 acres in Minnesota.

From 1887 to 1889 he held the position of Engineer in charge of Special Drainage District No. 1 of Onarga, Douglas, and Danforth Townships, Iroquois County, Illinois, on the drainage of 16 000 acres of land. From 1890 to 1901, he was engaged in private practice as Engineer for the drainage of large farms and for drainage districts organized under the laws of Illinois. In 1896, he prepared the first *Bulletin* on agricultural drainage issued by the United States Department of Agriculture.

In 1902, Mr. Elliott entered the Government service as Drainage Expert and, later, was made Chief of Drainage Investigations, in the Office of Experiment Stations, Department of Agriculture, where he continued to carry on the work until 1913. He was commissioned by the Secretary of Agriculture in 1908 to visit Northern Europe and to investigate and report on land drainage, with particular reference to the drainage of turf and peat lands in Holland, England, and Germany. He also made investigations and reported on plans for Minnesota drainage, and, in addition, reported on the drainage of prairie lands in Canada. During the period of his Government service Mr. Elliott personally developed the methods now generally used in the arid regions of the West for draining irrigated lands, and thus saved many irrigation projects from apparent failure.

After 1913, he was engaged in private practice as a Consulting Drainage Engineer and, at the time of his death, was a member of the Elliott-Harman Engineering Company, with offices in Peoria, Ill., Memphis, Tenn., and Washington, D. C.

Mr. Elliott was the author of "Practical Farm Drainage" (1882, 1908), and "Engineering for Land Drainage" (1903, 1911, revised in 1919), both of

* Memoir compiled from information on file at the Headquarters of the Society.

which have been widely used as textbooks; also of "Drainage of Farm Lands" (*Bulletin 187,* U. S. Department of Agriculture, 1904). At the meeting of the International Engineering Congress in San Francisco, Calif., in 1915, he presented a paper on "Drainage as a Correlative of Irrigation".*

Mr. Elliott was a pioneer in agricultural drainage and for many years was recognized as one of the leading American authorities in that branch of engineering. He was a Republican in his political sympathies, and a member of the Illinois Society of Engineers and the Washington Society of Engineers.

He was married on January 1, 1879, to Lura M., daughter of Leonard L. Bullock, of Normal, Ill., and is survived by a son, Herman R. Elliott, of Montrose, Colo.; a daughter, Mrs. P. B. Morehouse, of Washington, D. C.; and a sister, Mrs. Ida Knapp, of Deerwood, Minn.

Mr. Elliott was elected a Member of the American Society of Civil Engineers on September 3, 1890.

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* *Transactions,* International Eng. Congress, San Francisco, Calif., Paper No. 39, Waterways and Irrigation, p. 510.

OSWALD HERBERT ERNST, M. Am. Soc. C. E.*

DIED MARCH 21, 1926.

Oswald Herbert Ernst was born at Cincinnati, Ohio, on June 27, 1842. He was the son of Andrew Henry and Sarah (Otis) Ernst, his mother having been a daughter of George Alexander Otis of the distinguished Massachusetts family of that name. He entered Harvard College in 1858 at the age of sixteen, his education up to that time having been received at private schools. After two years at Harvard he received an appointment to the West Point Military Academy from the State of Ohio, and was admitted to that institution on July 1, 1860. He was graduated from West Point on June 13, 1864, and was promoted on that date to the rank of First Lieutenant, Corps of Engineers, U. S. Army.

Mr. Ernst's career at West Point was marked not only by high grade in academic studies but by characteristics of personal prominence in other respects. This is shown by the fact that during his last three years of cadet life he was selected to fill responsible positions in the military organization of the Cadet Corps. In his second year at West Point he was appointed one of the ranking Corporals; in his third year he was the First Sergeant of one of the four Cadet Companies; and during his entire fourth year he was First Captain of the Cadet Battalion.

The selection of cadets to fill the positions of officers in the Battalion, is made by the Academy authorities with the intent of securing those cadets most likely to inspire the confidence and command the respect of their cadet associates. The position of First Captain in the Corps at that time, and until quite recently, was the most responsible one entrusted to a cadet. The selection of the First Captain is based partly on soldierly bearing and partly on the confidence of the authorities in his ability, his good judgment, his impartial disposition, and his appreciation of the many responsibilities that devolve upon him. Mr. Ernst met all these requirements and expectations to the satisfaction of the authorities and with such calm, courteous, and impressive dignity that his associates dubbed him "Deacon" Ernst, a title which colleagues of cadet days ever after applied to him.

For the first fourteen years after his graduation he was engaged almost entirely on duty purely of a military nature. During the first year after graduation, as an Assistant Engineer of the Army of Tennessee, he was engaged in the battle and siege of Atlanta, and other active field operations. His other purely military occupation in these years embraced construction work on fortifications in the Harbor of San Francisco, Calif. For three years, from 1868 to 1871, he was an Instructor in the Engineering School of Practice at Willets Point, Long Island; and for seven years, from 1871 to 1878, he served as Instructor in Practical Engineering at the West Point Military Academy. During this latter period he was assigned the civil

* Memoir prepared by S. E. Tillman, Brig. Gen., U. S. A. (*Retired*), New York, N. Y.

detail of Astronomer of the United States Commission to observe the solar eclipse in Spain in December, 1870.

For the next fifteen years, 1878 to 1893, Captain (later, Major) Ernst's services were devoted to engineering and other works of a civil nature. These duties included improvements in the navigation of the Osage River, Missouri, and of the Mississippi and Missouri Rivers above their point of junction, at various places in Illinois and Missouri. The improvements extended also to the interstate harbors of these rivers. During part of this period, from 1886 to 1889, he was also in charge of the river and harbor improvements of Texas and from 1889 to 1893, of the Public Grounds and Buildings of Washington, D. C.

Captain Ernst received the brevet of Captain in March, 1865, for "faithful and meritorious service", within less than a year after his graduation. He attained the rank of Captain of Engineers in 1867; that of Major in 1882; and Lieutenant Colonel in 1895. In the period between 1881 and 1889 he was a member of the Mississippi River Commission and of various engineering boards, on river and harbor works and bridge construction. He reached the rank of Colonel of Engineers in 1903.

From 1893 to 1898, Major (later, Lieutenant Colonel) Ernst was in command at West Point and Superintendent of the Military Academy. In 1898 he left West Point to take part in the Spanish-American War, and commanded the First Brigade of the First Division of the First Corps in the Porto Rican Campaign in 1898 and 1899. When the war was over, he returned to engineering work and from 1899 to 1904, was member of a Commission to determine a route for a canal across Panama. From 1903 to 1906 he was President of the Mississippi River Commission and also from 1902 to 1905, Chairman of the Board of Engineers to survey a route for a 14-ft. waterway between Chicago, Ill., and St. Louis, Mo. During 1905 and 1906 he served as a member of the Isthmian Canal Commission. Colonel Ernst was retired from active service with the rank of Brigadier General on June 27, 1906, having reached the legal age of retirement.

After retirement he continued engineering work as a member of the International Waterways Commission until 1913. He was appointed to this body in 1903 and was thereafter Chairman of the American Section. From 1906 to 1908 he was Consulting Engineer of the East Side Levee Association for the protection of the American bottom opposite St. Louis, and prepared the plans therefor. In 1909 he was Consulting Engineer on plans for the drainage of the Coldwater-Tallahatchie Basin in the alluvial bottom of the Mississippi River. In 1914 and 1915 he was President of the Board of Consulting Engineers convened by the Miami Conservancy District to examine plans to protect the valley of the Miami River, Ohio, from the destructive effects of floods. For more than twenty years after his retirement from the Isthmian Canal Commission he rendered valuable service as a Director of the Panama Railroad Company (which operates the Steamship Line also), until only a few months prior to his death. In November, 1916, he was promoted to the rank of Major General on the retired list of the Army.

The personal characteristics of General Ernst were so fine and rare as to deserve special mention. He was of large and impressive physique, his manner was courteous and friendly, rather gentle but always dignified; his physiognomy, especially his eyes, and his calm, careful expression in conversation inspired ready confidence and brought conviction of his honest and frank nature. His mind was not rapid in operation, but extremely logical and sure in conclusion; his sense of and desire for justice were strong and his integrity was of the loftiest; his ideals of conduct and his sense of duty in all relations of life were very high and these he endeavored to inculcate by precept as well as example. His life is an illustration of beautiful conformity to the motto of his Alma Mater—Duty, Honor, Country—which motto he helped to frame. His record is one of long, honorable, distinguished, and useful service to his country.

He died on March 21, 1926, in his eighty-fourth year. His widow and two daughters, Helen Ernst and Mrs. William Morton Grinnell, survive him.

General Ernst was elected a Member of the American Society of Civil Engineers on July 4, 1888.

HERBERT CLARK FELTON, M. Am. Soc. C. E.*

DIED MAY 15, 1926.

Herbert Clark Felton, the son of Amory and Nancy Perkins (Boynton) Felton, was born in Troy, N. Y., on November 3, 1846. After completing his early education in the local schools he entered Rensselaer Polytechnic Institute, from which he was graduated in 1866 with the degree of Civil Engineer.

In the fall of that year, Mr. Felton entered the employ of the Novelty Iron Works, in New York, N. Y., as Draftsman, which connection he continued for two years. During this same period he taught Mechanical Drawing in the School of Science at Cooper Union Free Night School.

Late in 1868, he accepted an offer with Messrs. Brown Brothers and Company, Bankers, of New York, to undertake preliminary and location surveys for a number of railroad projects in which that Company was interested. That his services were satisfactory to his employers is attested by the fact that he remained with them for a period of twenty years. During the early part of his assignment, he was Superintendent of the Fort Lee Railroad and made a survey for a new railroad line along the west bank of the Hudson River, starting opposite New York. In 1872-73 he made a location survey for the proposed Camden and Manchester Railroad, extending from Camden to Manchester (now Lakehurst), N. J. Had it been built, this railroad would have given the New Jersey Southern (Central Railroad of New Jersey) entrance to Philadelphia, Pa.

On December 2, 1873, the Kaighn Point, Camden, and Philadelphia Ferry Company was purchased by Brown Brothers and Company, and Mr. Felton assumed the duties of Secretary, Treasurer, and Superintendent of this Company which operated ferry boats between Chestnut and South Streets, Philadelphia, and Kaighn Point, Camden. In 1888, the Reading Railroad purchased this Ferry Company from Brown Brothers and Company, retaining Mr. Felton in the capacity of Superintendent, which position he held until his retirement on February 1, 1917.

Mr. Felton was married on April 10, 1872, to Harriet Louisa Job, of Troy, N. Y., who survives him.

He was one of the organizers of the Camden National Bank, with offices in Camden and Philadelphia, having been a Director from the time it opened its doors for business on August 13, 1885. On January 13, 1905, he was elected Vice-President and continued in that capacity until his death. Mr. Felton was also a Director of the Atlantic City Railroad Company, the Port Reading Railroad Company, and, at the time of his death, was a member of the Chamber of Commerce of Camden and the Chamber of Commerce of Philadelphia.

He was an exceedingly jovial and kindly man, much beloved by all who knew him.

Mr. Felton was elected a Member of the American Society of Civil Engineers on June 1, 1887. He was also a member of the Society of Naval Architects and Marine Engineers.

* Memoir prepared by Henry J. Sherman, M. Am. Soc. C. E.

CLARENCE THAYER FERNALD, M. Am. Soc. C. E.*

DIED MAY 11, 1926.

Clarence Thayer Fernald was born in Charlestown, Mass., on November 10, 1866. He was the son of Ichabod Nichols and Abigail (Thayer) Fernald. He received his early education in the public schools in Charlestown, and later attended the Lowell Institute Courses of the Massachusetts Institute of Technology, specializing in scientific studies, advanced mathematics, etc.

In 1884, Mr. Fernald entered the employ of Mr. J. O. Goodwin, an engineer of Medford, Mass., as a Rodman and remained with him for six years, on general engineering and municipal work.

In 1890, he became connected with the Norfolk and Western Railway Company, as Transitman and Chief of Location Party, remaining with that Company until 1891, when he established, with an associate, a general engineering practice in Melrose, Mass. In 1894, he was appointed First Assistant Engineer in the Sewer Department of Melrose, and had charge of the survey and construction of 30 miles of its sewerage system.

Shortly after the Boston Elevated Railway Company began the construction of the Rapid Transit System in Boston, Mass., it secured the services of Mr. Fernald, his appointment being made in February, 1899. From that time, he had an intimate part in the development of the Rapid Transit System. He devoted the greater part of his twenty-seven years of service with the Boston Elevated Railway Company to this work, which consisted of designing and constructing elevated structures, viaducts, tunnels, subways, and, just prior to his death, the Dorchester Rapid Transit Extension over the right of way of the Shawmut Branch of the New York, New Haven and Hartford Railroad. Mr. Fernald was recognized as an authority on all the varied problems pertaining to rapid transit engineering, and the work which he did in connection with the development of the Rapid Transit System of Boston will always remain as a monument to his engineering ability. He was consistently painstaking and accurate in every detail of research, design and construction, and was also energetic in the carrying out of each project.

In the civic affairs of his home city—Melrose—Mr. Fernald took an active interest. He served for four years as Alderman at Large on its first Board of Aldermen, and, later, for seven years, as Chairman of the Board of Park Commissioners. In addition, he also served on many committees on various civic improvements.

In the Presidential Campaign of 1905, he was a member of the Massachusetts Electoral College, having been elected Presidential Elector from the Eighth Congressional District of Massachusetts, of which District he was for eight years a member of the Congressional Committee.

* Memoir prepared by H. M. Steward, M. Am. Soc. C. E.

Mr. Fernald was active in fraternal orders. He was a Past-Commander of Wyoming Lodge, F. and A. M., of Melrose, and a member of Waverly Royal Arch Chapter, Hugh de Payens Commandery, and Aleppo Temple, A. A. O. N. M. S. In the Melrose Lodge of Elks, he was Past-Secretary. He was also a member of the Boston Society of Civil Engineers, New England Street Railway Club, New England Traffic Club, and American Electric Railway Association.

Mr. Fernald was a man of sterling character, and of genial disposition, although he was positive in his convictions. He always enjoyed a wide circle of friends, through his affiliations with a large number of organizations, to which he freely gave of his time and where his presence and advice will be greatly missed. To those who had the pleasure of being associated with him in his work he endeared himself by his marked ability, loyal friendship, and willingness to offer any assistance he was able to give.

He was twice married and is survived by his widow and a daughter.

Mr. Fernald was elected a Member of the American Society of Civil Engineers on October 7, 1908.

CLARK ROBINSON FICKES, M. Am. Soc. C. E.*

DIED FEBRUARY 8, 1926.

Clark Robinson Fickes was born near Orrstown, Pa., on May 27, 1873. He received his education, both elementary and technical, in Iowa City, Iowa, and was graduated from the State University of Iowa in 1893 with the degree of Bachelor of Science. He received the degree of Civil Engineer in 1906.

During the period from 1894 to 1896, Mr. Fickes was Assistant City Engineer and City Engineer, of Iowa City, and designed and directed the construction of storm and sanitary sewers, and streets.

From 1896 to July, 1897, he was engaged as Assistant to R. M. Cannon, Civil and Mining Engineer, at Colorado Springs and Cripple Creek, Colo., computing mineral surveys and supervising the construction of a large ore-treating plant. At the request of the City Council, he then returned to Iowa City to take charge of the construction of three miles of pipe sewers.

In 1898, Mr. Fickes entered the employ of the Chicago, Burlington and Quincy Railroad Company, as Assistant Engineer, in which capacity he was engaged in general railroad maintenance until 1904. During this time he had direct charge of the construction of a large freight yard near Hawthorne, Ill., consisting of 75 miles of track, bridges, and other accessories; made all the preliminary surveys and was responsible for the double-tracking of the main line between Monmouth, Ill., and Burlington, Iowa, which included heavy work, due to grade and curve eliminations. He also had charge of the office work in connection with the double-tracking of the main line through Iowa and designed all minor structures in connection therewith.

From November, 1904, to March, 1905, Mr. Fickes served as Drainage Engineer for the Des Moines County, Iowa, Commissioners, making a survey and an estimate for the drainage of about 50 000 acres of swamp land.

During 1905 and 1906, he was employed as Resident Engineer for the St. Louis Southwestern Railway Company on the construction of seven bridges over the Red River, at Shreveport, La. This was considered a very troublesome and rather difficult piece of work on account of high-water conditions. Concrete caissons were carried to rock, by open dredging, through 85 to 90 ft. of clay and sand.

In 1906, Mr. Fickes again entered the employ of the Chicago, Burlington and Quincy Railroad Company as Assistant to the Bridge Engineer, in charge of maintenance and construction field work on all bridges on the System.

In 1912, the project of extending the Southern Illinois Line of the Burlington System across the Ohio River to a connection with the Nashville, Chattanooga, and St. Louis Railroad, at Paducah, Ky., was given its first impetus, and Mr. Fickes was sent to Metropolis, Ill., to lay down the bridge line, make the necessary surveys, soundings, etc. During the progress of this work, and on the death of C. H. Cartlidge, M. Am. Soc. C. E., Bridge Engineer of the Chicago, Burlington, and Quincy Railroad Company, and Chief Engineer of

* Memoir prepared by Harrison C. Shafferman, Esq., Miami, Fla.

the Paducah and Illinois Railroad Company, Mr. Fickes was appointed Chief Engineer and handled successfully all the duties in connection with the position. In the summer of 1917, when the steel work was just about completed on the Metropolis Bridge, he was compelled to resign on account of serious illness contracted during the sinking of the caissons under high pressure.

After regaining his health in the spring of 1918, Mr. Fickes accepted a position with the Valuation Department of the Interstate Commerce Commission, with headquarters at Chattanooga, Tenn.

In June, 1918, he was commissioned a Major of Engineers, U. S. Army, and ordered to France. During the remainder of the World War and until the spring of 1919, he was engaged on the construction of railroad and dock facilities at the proposed American port at Talmont, near the mouth of the Gironde River, where he was Commanding Officer of a camp of 4 000 men, including several companies of the 18th and 32d Engineers, labor battalions, and German prisoners.

On his return to the United States in the spring of 1919, Major Fickes was employed as Assistant to H. N. Rodenbaugh, M. Am. Soc. C. E., who was then Engineer of the Southern District for the United States Railroad Administration, with an office at Atlanta, Ga. In this capacity, he supervised the reconstruction of the Southern Railway Bridge across the Tennessee River near Chattanooga, Tenn., which work was accomplished in less than five months.

In March, 1920, when the Railroad Administration relinquished active charge of the lines, he was called to Washington, D. C., in connection with the work of liquidation, and in April of the same year, he was appointed Regional Director, with an office in Chicago, Ill. His work was carried out so successfully that his duties under that appointment were completed by October, 1921, after which he was called to Washington and given charge of the settlement of claims of about sixty small lines in various parts of the United States. This task was completed in July, 1922.

In June, 1923, Major Fickes accepted the position of Resident Engineer of the Florida East Coast Railway Company, in charge of the reconstruction of the St. John's River Bridge at Jacksonville, the project including the replacing of the single-track structure with double track of a much heavier and most modern type. Under his efficient direction, this bridge, which consisted of twenty-two girder spans, three fixed truss spans, a machinery span, and a 216-ft. Strauss heel trunnion bascule leaf span, of a total length of 2 451 ft., with twenty-six piers and two abutments, all pneumatically founded, was constructed at a cost of $2 500 000. The bridge was built at a time when both rail and river traffic was the greatest in the history of the State of Florida, with practically no delay to either.

He was a man of wonderful personality, greatly beloved by those who knew him, and as an engineer his word was authority on all matters pertaining to his profession. He will be greatly missed by his fellow members and by his many friends.

Major Fickes was elected a Member of the American Society of Civil Engineers on November 7, 1906.

SAMUEL BROWNLEE FISHER, M. Am. Soc. C. E.*

DIED JULY 9, 1926.

Samuel Brownlee Fisher, the son of Jacob Piper and Jane Thompson (Brownlee) Fisher, was born at Cherry Fork, Ohio, on October 22, 1846. He was a student at Washington and Jefferson College, Washington, Pa., from 1862 to 1868. His mental attainments brought him recognition from his Alma Mater and he received the degrees of Bachelor of Science in 1868, Master of Science in 1871, and Doctor of Science in 1908.

From 1869 to 1871 he was in the Rogers Locomotive and Machine Works, at Paterson, N. J., learning locomotive building. During the following year he was engaged in the locomotive repair shops of the Pittsburgh, Cincinnati, and St. Louis Railway Company, at Dennison, Ohio, and from 1872 to 1873, he held a position with the Pittsburgh Locomotive Works, at Allegheny, Pa.

From 1873 to 1886 Mr. Fisher was employed in the Engineer's Office of the Pennsylvania Company at Pittsburgh, Pa., and was engaged in various work, including preliminary and location surveys in Western Pennsylvania and Northeastern Ohio, and on second-track work. He was also engaged on the maintenance and construction of harbors on Lake Erie, sidings, the relocation of the Pittsburgh, Fort Wayne, and Chicago Railway and its branch lines, and on the construction of a low-grade line along Beaver River from New Brighton to New Castle, Pa.

From 1886 to 1890 he was Chief Engineer of the Milwaukee and Northern Railway Company at Green Bay, Wis. He surveyed and built the railway from Iron Mountain to Republic, and from Rockland to Sidnaw, in Michigan. He also superintended the location of the Green Bay Kewaunee Railway.

From 1890 to 1892 he was in the employ of the Minneapolis, St. Paul, and Sault Ste. Marie Railway Company and built the Hankinson Branch of that road. He also made a reconnaissance survey to the International Boundary.

Mr. Fisher was Chief Engineer of the Everett and Monte Cristo Railway from 1892 to 1894 and built a line from Puget Sound to the heart of the Cascade Mountain Range.

In the winter of 1894-95 he made a reconnaissance survey for the railway from the St. Croix River near St. Paul, Minn., to Ashland, Wis. Mr. Fisher was also engaged on construction work for contractors on a timber branch of the Chicago, Milwaukee, and St. Paul Railway among the Wisconsin lakes.

During 1895 and 1896 he served as Chief Engineer of the St. Louis and Kansas City Railway, and as Assistant Chief Engineer of the Missouri, Kansas, and Texas Railway. From 1896 to 1913 he held the position of Chief Engineer of the Missouri, Kansas and Texas Railway, with headquarters at St. Louis, Mo. From 1913 to 1915 Mr. Fisher was Chairman of the Valuation Committee, at St. Louis, and from 1915 until his death, he served as Consulting Engineer of the Valuation Department of this railroad, with headquarters

* Memoir prepared by A. W. Galbreath, M. Am. Soc. C. E.

in Parsons, Kans. The many improvements, extensions, and the excellent condition of the Missouri-Kansas-Texas System at the present time are due in a large measure to his work.

During his railroad career Mr. Fisher made countless friends in both business and railroad circles. He was a man of unusual ability and foresight and his retentive mind was a source of information that was considered reliable by his employers and the leaders in his profession. He was a born student and was continually adding to his intimate knowledge of a wide range of subjects of which those pertaining to his life's work were only a small part. At the age when most people consider their education finished, he was investigating new fields of knowledge. When sixty-nine years of age he attended night sessions in the Parsons High School to add to his knowledge the Spanish language. He was also deeply interested in religion.

While the Engineering Profession and Mr. Fisher's large circle of associates and friends have suffered a great loss in his passing, his sterling character, capacity, vision, and the example of his Christian life will be a continual source of inspiration to efforts for more useful and helpful lives.

He died at Parsons, on July 9, 1926, following an illness of a few days due to heat prostration. He was married on February 7, 1882, to Agnes Crooks, who died in December, 1906. He is survived by a son, Brownlee Fisher, of California, a daughter, Anne Fisher, of De Soto, Mo., and a grandson.

He was a member and an Elder of the Second Presbyterian Church of St. Louis for many years. He was also a Charter Member of the American Railway Engineering Association, and a member of the Engineers Club of St. Louis and of the St. Louis Railway Club.

Mr. Fisher was elected a Member of the American Society of Civil Engineers on October 7, 1903.

CHARLES FROMMER, M. Am. Soc. C. E.*

DIED OCTOBER 13, 1923

Charles Frommer, the son of Jacob and Mary Dorothea Frommer, was born in Belvidere, N. J., on December 5, 1873. Both his father and mother came from Germany and lived in Belvidere for many years. After his father's death, his mother moved to Philadelphia, Pa., where she provided a home for her family.

Mr. Frommer received his early education in the public schools of Belvidere, leaving his home town in 1888 to make his way in the world. In the same year he entered the employ of the Lehigh Valley Railroad Company, as a Rodman, and advanced, successively, through the positions of Chainman, Transitman, and Draftsman.

In 1892 he left the Lehigh Valley Railroad Company to accept a position with the Pencoyd Iron Company as a Structural Draftsman on the design of buildings and bridges. Mr. Frommer was with this Company a little more than two years until in September, 1894, he entered the employ of the City of Philadelphia as Draftsman in the Bureau of Surveys, where he advanced through the various grades.

In the fall of 1897 he enrolled in the night school of Drexel Institute, and was graduated in 1899, having taken the course in Building Design and Construction.

Mr. Frommer continued as a Draftsman in the Bureau of Surveys of Philadelphia until October, 1904, when he was advanced to the position of Assistant Engineer in Charge of Sewer Design. He continued in this work until January, 1916, when he was advanced to be Principal Assistant Engineer of the Bureau of Surveys. In this position he had direct contact with, and supervision of, the engineers who were in charge of the various divisions of the Bureau of Surveys. He continued as Principal Assistant Engineer until his death on October 13, 1923.

During the latter part of Mr. Frommer's service with the Bureau of Surveys, there was an interim of a year and one-half when the Bureau was without a Chief Engineer, and he was designated Acting Chief Engineer during that period.

Mr. Frommer was an ardent and indefatigable student and soon developed the qualities of a successful engineer. He was of a genial and generous disposition and was beloved and respected by those who came in contact with him.

On June 28, 1922, he was married to Katherine Muller, of New York, N. Y., who, with one child, a son, survives him.

Mr. Frommer was elected a Member of the American Society of Civil Engineers on June 4, 1913.

* Memoir prepared by H. T. Shelley, M. Am. Soc. C. E.

GEORGE MORRALL GADSDEN, M. Am. Soc. C. E.*

DIED APRIL 14, 1925.

George Morrall Gadsden was a native of Charleston, S. C., having been born there on October 19, 1865. His parents were Maj. Christopher S. and Eliza (Morrall) Gadsden, both of whom were of distinguished South Carolina families.

Mr. Gadsden was educated at the Citadel Military Academy at Charleston, having been graduated from that institution in 1886. He afterward entered the employ of the Pennsylvania Railroad Company as a Civil Engineer, remaining with it, at Pittsburgh, Pa., for two years. He then went to Savannah, Ga., and entered the employ of the Plant System, now the Atlantic Coast Line Railroad Company, of which his father was at that time a Vice-President. Subsequently, he resigned that position and engaged in phosphate mining in Florida, where he remained for two years. Having been compelled to leave Florida on account of his health, he rejoined the Plant System as a Civil Engineer. After serving in this capacity for several years, he was elected Director of Public Works of the City of Savannah, and through his excellent work as the head of that Department during several City Administrations, as well as in his other activities, he became a prominent and influential member of the community.

After severing his connection with the Department of Public Works, Mr. Gadsden organized and became the head of the Gadsden Contracting Company. This firm built the sea wall at Tybee Island, Georgia, the Atlantic Coast Line Railroad Bridge over the Altamaha River, and acted as Contractor for all the bridges, on the celebrated road to Tybee Island, over many miles of marshes. Under Mr. Gadsden's direction, his firm also acted as Contractor for many other important projects in the Southeast.

He was a veteran of the Savannah Volunteer Guards, which was organized in 1802 and is one of the oldest military organizations in the country. He organized Company D and was its first Commander. He was a member for many years of the Oglethorpe Club, the social club of Savannah; of Ancient Landmark Lodge, F. and A. M., and of the Benevolent and Protective Order of Elks. He was also identified with the banking interests of Savannah as a Director of the Exchange Bank.

Mr. Gadsden is survived by his widow, two sisters, and one brother, Philip M. Gadsden. His sisters reside in Charleston and his brother in Philadelphia, Pa. His funeral service was held from Christ Church, Savannah, and was attended by his numerous friends; the City flags were displayed at half-mast in his honor.

* Memoir prepared by George T. Cann, Esq., Savannah, Ga.

While modest and retiring, George M. Gadsden was a man of acknowledged ability in his profession and of the highest integrity in every phase of his life, a faithful friend, and most popular with all who knew him. Not only his friends but the community in which he lived sustained a distinct loss by his death.

Mr. Gadsden was elected a Member of the American Society of Civil Engineers on May 4, 1898.

VLADIMIR VASILLIEVICH GORIACHKOVSKY, M. Am. Soc. C. E.*

DIED AUGUST 9, 1926.

Vladimir Vasillievich Goriachkovsky was born at Kiev, Russia, on July 12, 1869. After graduation from the University of Kiev, he entered the Technical Institute in Petrograd, Russia, of the Ministry of Ways and Communications, from which he was graduated with the degree of Civil Engineer in 1897.

Mr. Goriachkovsky's first work in engineering was as Resident Engineer on the construction of the Sura River Bridge for the Moscow-Kazan Railway and, later, as Engineer in the Technical Department.

In January, 1903, he was appointed Resident Engineer by the Moscow-Windau Rybinsk Railway, in charge of grain elevator construction and harbor improvements. Later, he became Superintendent of Maintenance of Way, and Assistant to the General Manager. He remained in the capacity of Advisory Engineer to this railway until control of the road was lost by its owners.

From 1903 to 1910 Mr. Goriachkovsky made many trips abroad to study modern methods in engineering work, including the United States and Canada; Switzerland, during the driving of the Simplon Tunnel; Antwerp, Belgium, during the enlargement of the port; Roumania, during the construction of the grain elevator program; and England and Germany, to familiarize himself with cold storage construction and operation.

In 1910 Mr. Goriachkovsky was appointed Chief Engineer of the Altai Railway which was to open that most fertile valley in Siberia—a valley also rich in precious and semi-precious minerals.

As Engineer for the International Bank of Commerce of Petrograd, he built and operated a cold storage plant in that city and was appointed as the Russian Associate Engineer to co-operate with D. L. Hough, M. Am. Soc. C. E., in the development of the Moscow Central Stations Project which the bank and its group had undertaken to finance. In addition to his other activities, he was Associate Engineer and Director of the Arozdayou, Caucasus, Cotton Growers Company and had charge of the construction of its comprehensive irrigation system. He was also Controlling Consulting Engineer to the Russian State Bank for its Grain Elevator Department.

The climax of his career in his native country came during the World War when it became necessary to reach by rail an ice-free port on the Murman Coast of Lapland, the final goal of the Gulf Stream. The necessity for this road was perhaps the most serious engineering matter to be solved at that time by the Russian Government and Mr. Goriachkovsky was chosen as Chief Engineer and General Manager to build 1 000 km. of railroad suitable for heavy traffic through an uncharted territory where vast swamps were encountered, and during two Arctic winters. He accomplished the task in eighteen months. For this achievement he received the highest recognition of his own Government, and was decorated with the Cross of St. Michael and

* Memoir prepared by D. L. Hough, M. Am. Soc. C. E.

St. George by the British Government and made an Officer of the Legion of Honor of France. He had as many as 80 000 workmen employed on this undertaking to whom it was necessary to transport food, equipment, and tools, which in itself was an Herculean task.

Of this achievement the *London Times* wrote on October 30, 1915:

"When on the shores of the Arctic Ocean the first locomotive whistles, Russia will have the right to say that she has once more accomplished a labour of Titans and will be able fully to value the iron will, solid knowledge, and indefatigable energy of the Chief Constructor of the Murman Railway, Engineer V. Goriachkovsky, and his co-workers, great and small."

Mr. Goriachkovsky was not a specialist in one branch of engineering, but in all branches, and had been soundly instructed, as his accomplishments give proof. He was considered to be one of the first, if not the first, engineer of Russia. To these technical qualities were added geniality, tact, and every social gift. He was an entertaining speaker, an interesting companion, and a good friend.

Leaving Russia as late as May, 1918, he at once volunteered his services to the United States War Department, in Washington, and served in the Intelligence Department until the end of the World War. Subsequently, he became associated with the Youroveta Home and Foreign Trade Company for which he designed a chain of packing houses and cold storage plants it was hoped to establish in Siberia.

On March 5, 1919, he presented an interesting paper before the Society entitled "Engineering Construction in Russia and General Conditions in That Country", illustrating his remarks with lantern slides.

In 1920 Mr. Goriachkovsky started on a comprehensive trip in behalf of certain countrymen who wanted his advice as to a suitable location or locations for the colonization of Russian refugees. He visited the Southwestern United States, Mexico, the West Coast of South America, and crossed to Argentina, but in Buenos Aires his studies were cut short by a nervous breakdown. Crossing directly to Europe, he entered a sanitarium in Germany where he apparently recovered from his illness. He then made his home near Lausanne, Switzerland, where he died on August 9, 1926. He was buried in Genoa, Italy. His widow, Xenia Michaelovna, survives him, as well as a large family of brothers and sisters.

Mr. Goriachkovsky was elected a Member of the American Society of Civil Engineers on November 25, 1919.

JAMES MADISON GRAVES, M. Am. Soc. C. E.*

DIED JULY 23, 1926.

James Madison Graves was born on August 18, 1878, at Lexington, Ky., the son of James Madison and Adeline (Allen) Graves. After passing through the public schools of Lexington, and graduating from the High School, he entered the University of Kentucky at Lexington, from which he was graduated in 1900 with the degree of Mechanical Engineer. He was a member of the Sigma Chi Fraternity.

During his summer vacations at college, Mr. Graves worked as a machinist in order to gain practical experience in shop practice, and after graduation he was employed for about a year by the firm of Field and Hinchman, Consulting Engineers, of Detroit, Mich. From Detroit, he moved to Pittsburgh, Pa., where he was engaged as a Draftsman with the firm of Heyl and Patterson.

In 1903, he was employed as Assistant to the Chief Engineer of the Allegheny County Light Company, and remained as an employee or officer with this Company and its successor, the Duquesne Light Company, continuously until his death.

In 1906, Mr. Graves was appointed Superintendent of Power Stations in charge of the operation of the Company's two generating stations at 13th Street, Pittsburgh, and at Rankin, Pa., which at that time had a combined capacity of 12 750 kw. He continued to hold this office when the Company was taken over by the Duquesne Light Company in 1913, and to operate the generating stations throughout the period of rapid growth of the Company until July 1, 1920, when he was appointed Assistant General Manager, with broader duties and responsibilities. On December 1, 1922, he became General Manager of the Duquesne Light Company, and on January 1, 1925, he was promoted to the position of Vice-President and General Manager.

During his service with the Duquesne Light Company, Mr. Graves collaborated with its Engineering Department and its Consulting Engineers in the design and construction of additional generating station capacity, including the Brunot Island and Colfax Power Stations. This resulted in increasing the generating capacity of the company from 12 750 to 324 900 kw. At the time of his death the Company had under construction an additional capacity of 80 000 kw. at the Colfax Power Station, which, when completed, will make the total capacity of this station 270 000 kw. He also saw the number of customers of the Company increase, during his period of service, from about 14 000 in 1903, to about 250 000 at the time of his death. As Vice-President and General Manager he had under his supervision, in addition to the power stations, about 900 circuit miles of high-tension transmission lines, 200 sub-stations, 16 000 circuit miles of low-tension distribution lines, and 120 miles of underground conduit.

Mr. Graves took an active interest in the so-called "super-power" movement and the interconnection of electric systems, which was in progress at

* Memoir prepared by Maurice R. Scharff, M. Am. Soc. C. E.

the time of his death, and during the last year of his life saw the completion of the 66 000-volt tie-lines for interchange of power, connecting the system of the Duquesne Light Company with the systems of the West Penn Power Company on the east, and the Pennsylvania-Ohio Electric Company on the west, constituting the last links in a continuously interconnected power system extending from Chicago, Ill., to Boston, Mass.

"Jim" Graves, as he was known to his many friends, was a deep student of the electric industry and a tireless worker, who nevertheless found time to know, personally, almost every one of his fellow employees, to take an active part in the civic and social life of his community, and to win the admiration and respect of those with whom he came in contact. The splendid public service he had rendered made his sudden death a distinct loss to the entire Pittsburgh District. This was recognized on every hand by public bodies and individuals, whose expressions of regret were summed up in the words of Mr. A. W. Robertson, President of the Duquesne Light Company, "we have just lost one of the finest men who ever lived."

For more than ten years Mr. Graves was an active member of the Prime Movers Committee of the National Electric Light Association, and, during the World War, served the War Department as a member of the Power Board of the Pittsburgh District.

On March 29, 1913, Mr. Graves was married to Helen Ayres, of Pittsburgh, Pa. He is survived by Mrs. Graves, two children, Eleanor, and James M., Jr.; two brothers, Buford and George Graves, of Pasadena, Calif., and one sister, Mrs. George C. Webb, of Lexington, Ky.

He was a member of the Baptist Church; Fellowship Lodge No. 679 F. and A. M.; the Engineers' Society of Western Pennsylvania; the American Institute of Electrical Engineers; the American Society of Mechanical Engineers; the Pittsburgh Chamber of Commerce; the University Club of Pittsburgh; and the Fox Chapel Country Club.

Mr. Graves was elected a Member of the American Society of Civil Engineers on January 15, 1923, and took an active part in the affairs of the Pittsburgh Section of the Society.

WILLIAM HOOD, M. Am. Soc. C. E.*

DIED AUGUST 26, 1926.

William Hood, the son of Joseph Edward Hood, an editorial writer and part owner of the *Springfield Republican,* and Maria (Savage) Hood, was born in Concord, N. H., on February 4, 1846.

From his eighth to his sixteenth year, he attended public schools in Boston and Springfield, Mass. Shortly after the commencement of the Civil War, he enlisted as a private in Company A, 46th Massachusetts Volunteers, and remained in the service, taking part in a number of engagements, until shortly after the Battle of Gettysburg. After being mustered out, he retured home to complete his education which, to that time, had been preparatory to a classical course. His ambition, however, was to be an engineer, and in pursuance of this goal he entered the Chandler Scientific School of Dartmouth College and was graduated therefrom in 1867 with the degree of Bachelor of Science.

The building of the Union Pacific and Central Pacific Railroads, which was then in progress, attracted Mr. Hood as offering a promising field for the application of his engineering education. He went to California where he was employed as a Rodman on the Donner Lake-Summit Section of the Central Pacific Railroad, then under construction. His native ability as an engineer and his energy and devotion to duty soon won recognition and, in 1868, he was appointed Assistant Engineer on the construction of the line through Nevada and Utah, on which work he continued for nearly two years, until the connection with the Union Pacific Railroad was made at Promontory, Utah, in May, 1869. He was then transferred to the Sacramento Valley and was engaged, until June, 1872, on the construction of the Central Pacific Branches northward to Redding and southward to Goshen, Calif.

In 1872, the Southern Pacific Railroad Company, which had been incorporated in 1865, began the location and construction of its line through California, Arizona, New Mexico, and Texas; and Mr. Hood, as Assistant Engineer, was employed on the location and construction of the line between Goshen and El Paso, Tex. It was during this period that he located the Tehachapi Loop, one of the first, if not the first, example of its type in the United States. Several locations had been made on this section by others, but all had proved prohibitive in cost. Mr. Hood's solution of the problem, by spirally curving in the descent and tunneling under the part of the road already passed over, enabled the Company to select smoother ground and to obtain a shorter line with less curvature, at a lower cost than was otherwise possible.

In August, 1875, Mr. Hood was appointed Chief Assistant Engineer of the Southern Pacific Railroad Company, in charge of the construction through the Mojave and Colorado deserts and through Arizona and New Mexico, to San Antonio, Tex. In addition to the natural difficulties of the

* Memoir prepared by the following Committee of the San Francisco Section: Jerome Newman and W. H. Kirkbride, Members, Am. Soc. C. E.

country, which was practically unsettled, there was the presence of hostile Indians, against an attack by whom the engineers were always on guard.

On the completion of the Southern Pacific Railroad, Mr. Hood, in June, 1883, was appointed Chief Assistant Engineer, and in August of the same year, Chief Engineer, of the Central Pacific Railroad Company.

The Southern Pacific Company, Pacific System, which leased and operated all Central Pacific and Southern Pacific lines west of El Paso, Tex., was organized in 1885, and Mr. Hood was appointed its Chief Engineer, continuing in this position until June, 1900, when he was made Chief Engineer of the Southern Pacific Company, in charge of all lines extending from New Orleans, La., to Portland, Ore., and from San Francisco, Calif., to Ogden, Utah. He held this office until his retirement on May 4, 1921, on which date he had rounded out fifty-four years of service with the same Company.

During the period from 1883 to 1900, much important new construction was carried out under his direction. The most difficult work was the line between Redding, Calif., the original northerly terminus of the Central Pacific, and Ashland, Ore., the southerly terminus of the line from Portland. This line, which completed the mail route between San Francisco and Portland, ascends the Sacramento River Canyon to its head and crosses the Mt. Shasta Divide and the Siskiyou Mountains, the upper part being noteworthy for its steep grades, numerous tunnels, and curved switchbacks.

After the late E. H. Harriman became President of the Southern Pacific Company in 1901, the Company, under his enlightened policy of reducing operating costs, entered an era of improvement of the main lines constituting the System, particularly the Central Pacific, and to Mr. Hood was delegated the task of carrying out this work, in addition to which he was also consulted on important matters affecting other Western roads under the Harriman control.

One of the most notable achievements during the reconstruction of the Central Pacific was the completion of the Lucin Cut-Off across Great Salt Lake in Utah. The old line around the north shore of the lake was long and crooked with heavy grades; Mr. Hood's line struck boldly across the lake, on embankments where possible, and on trestle where the depth of water was great, resulting in a saving of distance of 44 miles, in curvature of 3900°, and in ascents of 1500 ft. The adoption of this route was at first strongly opposed by engineers of other railroads under the Harriman control, but Mr. Hood's arguments in favor of it were convincing enough to win their approval, and the work was carried out as planned.

Other important works for which Mr. Hood was responsible are the Dumbarton Cut-Off, crossing the southerly part of San Francisco Bay and affording rail connection for freight from the East to San Francisco; the second track over the Sierra Nevada Mountains from Rocklin to Blue Canyon, Calif., and from Truckee, Calif., to Sparks, Nev.; the Bay Shore Cut-Off from San Francisco to San Bruno, Calif., replacing the old heavy grade line between the same points; and the San Diego and Arizona Railway, connect-

ing the Imperial Valley with San Diego, Calif., which is noteworthy for its difficult tunnel work.

When Mr. Hood entered the service of the Central Pacific in 1867, that Company operated 90 miles of track, from Sacramento, Calif., eastward; when he retired in 1921, the mileage of the Southern Pacific Company, the successor of the Central Pacific Company, was 12 800, extending through Oregon, California, Nevada, Utah, Arizona, New Mexico, Texas, and Louisiana, with 1 240 miles in Mexico, nearly all of which was built under his direction.

William Hood was in many respects a remarkable man. Of robust constitution, he was physically well fitted to perform the work which first won him recognition as an excellent locating engineer; he had also an excellent "eye for country" and a most retentive memory for the physical characteristics of any section through which he had passed. It was his habit, when sending out a party to make a location survey, to give the engineer in charge a minute description of the country to be traversed, the possible sources of water supply, material for ballast, and other details of interest. Although he might not have visited the vicinity for many years, these descriptions were found to be accurate and aided materially in expediting the progress of the survey. He insisted on accuracy in field work, and required preliminary lines to be run with the same care as the final location.

He was a man of intense energy, working always at high pressure and for long periods. Age did not diminish his powers, and his mind was as keen and active at seventy-five as that of a man of forty. Loyalty to his tried assistants was one of his characteristics, and he had always, even when work was not plentiful, an engineering organization from which he could select the men best adapted for the work in hand. His assistants respected him for his ability and for that reason were glad to serve under him, knowing that good work on their part would be understood and appreciated, even if no compliments were paid.

Mr. Hood also enjoyed a reputation for fairness in dealing with contractors that was instrumental in enabling him to have his work done well and at reasonable cost. Knowing that they would be treated fairly, the firms would often take contracts at prices fixed by him, trusting him to make good any losses incurred by circumstances beyond their control.

Although Mr. Hood was first of all an engineer, he was also interested in other things. His principal hobby was photography, both in black and white and in color, and in pursuit of suitable subjects he was to be found every week-end climbing hills and tramping over the level country in the vicinity of San Francisco. These outings not only furnished needed recreation, but served likewise to keep him in excellent physical condition. As with everything that he undertook, he made a thorough study of photographic theory and practise and produced superlative pictures, which he was fond of distributing among his friends. He was also a skilled pianist, and a constant attendant at the numerous concerts in his home city.

In recognition of the important part played by Mr. Hood in the railroad development of the Pacific Coast and of his breadth of vision and experience as an engineer, Dartmouth College, in 1923, conferred upon him the degree of Doctor of Science.

Mr. Hood was married three times, the last time on September 13, 1923, to Mrs. Lucia O. Getzler, who, with a daughter, Mrs. Jessie Wetzel, and two sisters, Mrs. Anna M. Hall and Mrs. Florence Hood Dillon, survives him.

He was a member of many clubs, among which were the Pacific Union, Bohemian, Engineers, and Press Clubs of San Francisco, and the Jonathan Club of Los Angeles, Calif. He was also a member of the American Railway Engineering Association, Society of American Military Engineers, and the Astronomical Society of the Pacific, as well as a Fellow of the American Association for the Advancement of Science.

Mr. Hood was elected a Member of the American Society of Civil Engineers on October 7, 1896.

FRANCIS EDWIN HOUSE, M. Am. Soc. C. E.*

DIED APRIL 3, 1926.

Francis Edwin House, the eldest of a family of four sons of Henry A. and Mary E. (Goff) House, was born at Houseville, N. Y., on November 15, 1855. The family was of New England ancestry, and had founded the village early in the Nineteenth Century.

After a college preparatory course at a private school in Rochester, N. Y., Mr. House entered Rensselaer Polytechnic Institute, at Troy, N. Y., where he studied Civil Engineering and Chemistry, but was not graduated. He was a member of the Delta Phi Fraternity.

He left Rensselaer in 1877, in his Senior year, and went to Austin, Nev., then one of the important silver mining camps of the West. Here, for three years, he worked as Mining Engineer and Assayer for the principal mine of that camp. In Austin he also began the work with which he was to be identified for the remainder of his life—railroad engineering. Toward the end of his stay in Austin, in addition to his mining work, he completed the surveys and engineering work for the narrow-gauge mountain railway, connecting Austin with Battle Mountain, Nev., ninety miles distant.

In 1880, Mr. House entered the employ of the Chicago, Milwaukee and St. Paul Railway Company, and during the following three years was engaged in preliminary and location surveys for that railroad, the Wabash, and others, on extensions of their Systems in Missouri, Iowa, and Nebraska. He was made Division Roadmaster of the Chicago, Milwaukee and St. Paul Railway in November, 1883; General Roadmaster in 1887; and Trainmaster of the Kansas City Division in 1890.

In 1891 Mr. House joined the Engineering Staff of the Lake Shore and Michigan Southern Railway Company, and was engaged on construction for that System until in 1892 he went to the Pittsburgh and Lake Erie Railway Company as Engineer of Maintenance of Way. In 1894 he was promoted to the position of Chief Engineer.

In 1896 Mr. House went, with his superior, to build the Butler and Pittsburgh Railroad for the Carnegie Steel Company, and shortly afterward became Chief Engineer of the entire Pittsburgh, Bessemer and Lake Erie Railway, having charge of all improvements on the old line, as well as the building of the new part, which was notable on account of the heavy work required in its construction.

The late Andrew Carnegie, then active head of the Company, was much pleased with Mr. House and his work, and his advancement was rapid. He was made General Superintendent of the road in 1897, and General Manager in 1901. His superior officer in the organization at that time writes:

"If the Carnegie Steel Company had continued its independent operation, I think he would have gone up in that organization in some other department than its railroad."

* Memoir prepared by a Committee of the Duluth Section, consisting of Messrs. W. A. Clark, W. H. Hoyt, and O. H. Dickerson, Members, Am. Soc. C. E.

Upon the formation of the United States Steel Corporation in 1901, Mr. House went to Duluth, Minn., as President and General Manager of the Duluth and Iron Range Rail Road Company, which position he held continuously until his death, on April 3, 1926, except that during Federal control of the railroads under the United States Railroad Administration, he served as Federal Manager at the same time of both the Duluth and Iron Range Rail Road, and the Duluth, Missabe and Northern Railway.

He was endowed with a splendid physique, and as an executive showed remarkable energy and great capacity for work, a contagious, almost boyish enthusiasm, and a very high ideal of justice and fair dealing. Although a believer in strict discipline, justice was tempered with mercy in his decisions, and their fairness was seldom questioned.

Mr. House was a remarkably fine type of man and citizen; devoted to his home and family; a loyal friend and a delightful associate; liberal in his support of worthy causes; of broad interests; keen in business; and remarkably successful as an engineer and executive. He served as an Elder in the Presbyterian Church for more than twenty-five years, and was active in its work. For nearly eighteen years he was a Director of the Duluth Young Men's Christian Association, for ten years its Vice-President, and its Acting President for two periods, amounting to about three years. He was also a member of the National War Work Council of the Association during the World War.

In 1918 his Alma Mater conferred on him the Honorary Degree of Master of Civil Engineering. Mr. House was the first and only person to have received it.

Mr. House was married in 1878 to Mary V. McCracken, at Des Moines, Iowa. His wife and four of six children survive him.

Mr. House was elected a Member of the American Society of Civil Engineers on May 1, 1895, and although for the last thirty years of his life he was not directly engaged in engineering work, he retained an active interest in engineering activities. The Duluth Section of the Society is greatly indebted to him as a leader in its organization and as its first President.

AUGUSTUS SAYRE KIBBE, M. Am. Soc. C. E.*

DIED AUGUST 21, 1926.

Augustus Sayre Kibbe was born in Albany, N. Y., on August 8, 1865, the son of Augustus Kibbe, whose ancestors came from Exeter, England, in 1611, and Sarah A. (Sayre) Kibbe, of French descent, whose family came from Rouen, France. As a boy he lived in Brooklyn, N. Y., and attended the Polytechnic Institute of Brooklyn, later going to Troy, N. Y., where he received the degree of Civil Engineer from the Rensselaer Polytechnic Institute in 1886.

Upon graduation Mr. Kibbe was Assistant to the Director of the Rensselaer Polytechnic Institute for one year, after which he became Assistant Engineer with the State Engineer of New York, in charge of the Champlain Canal enlargement, with headquarters at Fort Edward, N. Y. . After two years on this work he was appointed Special Assistant to the New York State Engineer and Surveyor in August, 1888, with headquarters at Albany, N. Y., and was engaged in a number of important undertakings in connection with State surveys, water supply, and regulation, including the Black River and Canal water supply, the Genesee River water supply and storage, Adirondack reservoirs, New York maps for the Municipal Consolidation Commission, New York-Canada and New York-Pennsylvania boundaries, etc.

In April, 1892, Mr. Kibbe resigned from the New York State Engineering Department to engage in electric railway work as Engineer and Superintendent of the Woodbridge and Turner Engineering Company, in New York, N. Y. While with this firm he designed and constructed a number of electric railways, among which were the Paterson Central Railroad of New Jersey, the Washington, Alexandria and Mount Vernon Railway, the Brigantine (N. J.) Transit Company's Railway, the Chester (Pa.) Traction System, and parts of the People's Traction Company of Philadelphia, Pa. He severed his connection with the Woodbridge and Turner Engineering Company in October, 1895, to become Engineer of Construction of William Wharton, Jr., and Company, with which Company he remained for four years. While with this Company he built the Fairmont Park Transportation Company's Railway, Philadelphia.

Commencing in June, 1899, as Engineer for the American Railways Company, Philadelphia, Mr. Kibbe became Chief Engineer of that Company in 1904, which position he held until November 30, 1910. During this time he had engineering charge of the construction, equipment, and operation of the electric railway, lighting, and power plants of the American Railways Company, which comprised about 350 miles of track and a capacity of 15 000 kw. in power stations, located in various cities of the Middle Western States. In 1911, he opened offices as a Consulting Engineer in Chicago, Ill., but in 1912, transferred his activities to Reno, Nev. From 1914 until his death Mr. Kibbe

* Memoir prepared by George H. Binkley, M. Am. Soc. C. E. and J. Otis Burrage, Esq.

was with the Key Transit System, Oakland, Calif., as Consulting Engineer in charge of valuation and rates. During this time he also served as Consultant in valuation matters for the Northern Electric Company and the Sacramento-Northern Railway of California.

Outside his engineering work, Mr. Kibbe's great interest was in wild life conservation and he was Secretary of the California Societies for the Conservation of Wild Life. For six years previous to his death he served as President of the Audubon Association of the Pacific. Probably no bird lover was better informed as to California birds, and he was considered an authority both on the knowledge of birds and their protection, which was the purpose of the Association. He was also a member of the Sierra Club and one of the original members of the Pacific Railway Club.

Mr. Kibbe was always a student and a man of high ideals and character. He was quiet, modest, and retiring, but with his keen, analytical mind he arrived at his decisions and stood firmly for them in his unassuming manner. He was loved and respected for his faithfulness, loyalty, and honesty by all with whom he came in contact.

He left a son, Augustus Sullivan Kibbe, of West Chester, Pa., and a daughter, Helen Towne, of Dayton, Ohio. His widow, Bessie W. Kibbe, resides in Berkeley, Calif.

Mr. Kibbe was elected a Junior of the American Society of Civil Engineers on March 6, 1889, and a Member on August 31, 1909.

ZAC ELLIS KNAPP, M. Am. Soc. C. E*

DIED OCTOBER 1, 1926.

Zac Ellis Knapp was born at Edenton, N. C., on September 12, 1867. After receiving his preparatory education, he attended the Colorado State School of Mines at Golden, Colo., during the sessions of 1884-85 and 1885-86, taking the first two years' course in Civil and Mining Engineering. Leaving college, he continued his studies at his home in Grand Rapids, Mich., being engaged, at the same time, in surveying in or near that city.

In 1888, Mr. Knapp obtained a position as Levelman on a preliminary survey with the Grand Rapids and Indiana Railroad Company and in 1889 served as Transitman on preliminary and location work on the Middle Georgia and Atlantic Railroad from Savannah to Atlanta, Ga. From 1890 to 1891 he held the position of Assistant with Morris and Darley, Engineers, at Bristol, Tenn., and from 1892 to 1893 he was in the City Engineer's Office at Grand Rapids, Mich.

In 1894, Mr. Knapp went to Chicago, Ill., where he accepted the position of Chief Draftsman in connection with the electrification of the North and West Chicago Street Railways, the Lake Street Elevated Railway, the Northwestern Elevated Railway, the Union Loop, and allied lines. Later, he became Principal Assistant in charge of power-house construction, his work including the construction of a power-house for the Northwestern Elevated Railway, as well as alterations to existing power-houses.

Having resigned his position in Chicago, in 1901, Mr. Knapp became associated with Mr. James R. Chapman. They went to England together, Mr. Chapman as General Manager and Chief Engineer, and Mr. Knapp as Assistant Engineer, for the Underground Electric Railways Company of London, Limited. Their work consisted of the electrification of the Metropolitan District Railway, the building of three tubes (now the London Electric Railway), and the design and construction of a power-house and twenty-three substations, as well as car sheds. Mr. Knapp was also in responsible charge of the design of the steelwork for the passenger stations for the tubes.

He held this position until 1910 when he became Assistant General Manager of the London United Tramways, which was taken over by the Underground Electric Railways Company, with Mr. A. H. Stanley (now Lord Ashfield), as General Manager. On July 26, 1911, Mr. Knapp was appointed General Manager of the London United Tramways. During his administration, he made many important improvements in the organization and introduced new methods in staff control. In the early part of 1913, he resigned this position to return to the Underground Electric Railways Company as Chief Engineer, and, in 1915, he was appointed Manager for Maintenance and Construction.

* Memoir compiled from information on file at the Headquarters of the Society.

During the World War, Mr. Knapp took a leading part in plans for helping the dependents of employees who were enrolled with the military forces. On March 12, 1917, he became affiliated with the Associated Equipment Company, as General Manager of the Walthamstow Works, and was engaged in supplying munitions in the form of lorries, shells, etc.

In 1918, he returned to the Underground Electric Railways Company as Manager for Maintenance and Construction and, for a short time, as Commercial Manager. In 1921 Mr. Knapp was appointed Director of Construction in responsible charge of the engineering work pertaining to the reconstruction of the City and South London Railway and the extension of that line to Morden, as well as the Edgware and the Kensington Extensions of the Hampstead Tube, the Camden Town Junction, and other works. He held this position at the time of his death.

The loss of his wife to whom he was deeply attached, in June, 1925, together with the unsparing manner in which he devoted himself to his duties as Director of Construction, undoubtedly hastened his death.

Mr. Knapp was held in high esteem by the officials of the companies with which he was affiliated and had endeared himself to the employees by his kindly sympathy and consideration for their welfare.

He was a member of the Institute of Transport and of the Pilgrim Society, and was a naturalized British subject.

Mr. Knapp was elected a Member of the American Society of Civil Engineers on December 5, 1906.

WILBUR FISK McCLURE, M. Am. Soc. C. E.*

DIED JUNE 22, 1926.

Wilbur Fisk McClure was born in Perryville, Ohio, on December 13, 1856, the second son of Thomas Clarendon and Sarah A. (Stephens) McClure. His early childhood was spent in Ohio and Pennsylvania where his father held various charges as an ordained Minister of the Methodist Episcopal Church in the days when "itineracy" meant moving at least every two years. He attended the Southwestern Normal School at California, Pa., and on leaving this institution taught in the public schools of Fayette and Washington Counties, Pennsylvania, at Carthage, Mo., and in the Cherokee Nation.

While engaged as a teacher Mr. McClure took up the study of engineering, for which he left the Teaching Profession early in 1879, beginning work with the St. Louis and San Francisco Railroad Company as an Assistant Transitman on location. From November, 1879, to June 1, 1882, he was in the employ of the same Company as Assistant Engineer in charge of construction, or as Assistant on location in Missouri, Kansas, Arkansas, Indian Territory, and Texas. During the last six months of 1882, he was in charge of the Land Department of the Company, surveying lands and town sites.

In March, 1883, Mr. McClure went to Deming, N. Mex., as Assistant Engineer in charge of tracklaying on the Deming, Silver City, and Pacific Railroad (narrow-gauge). On the completion of this road in the same year he moved to Los Angeles, Calif., and from 1883 to 1886 was engaged in private practice, chiefly in surveying oil claims for the Pacific Coast Oil Company and acting as Engineer for the Mountain Water Company.

From 1887 to 1893, he served as Chief Engineer of the Los Angeles Terminal Railway System (now the Union Pacific in California). In 1893 he went into Mexico where he surveyed and reported on mining property for Mr. T. B. Burnett and others of Los Angeles. While on this work he contracted malarial fever and was brought home to Los Angeles, where he lay seriously ill for three months. The effects of this illness, in fact, never entirely left him.

While convalescing at his home in Los Angeles, Mr. McClure received a call to the Methodist ministry and for the next seven years he served as a lay missionary in that region in California lying mainly east of the Sierra Nevada, with successive charges in Greenville, Plumas County; Bishop, Inyo County; Cedarville, Modoc County; and Truckee. At that time this was a pioneer, isolated field, and he is still affectionately remembered there for his sincere and kindly spiritual leadership.

As ill health had forced Mr. McClure out of engineering in 1893, so in turn it forced him out of the ministry and back to engineering in 1900. In that year he moved his home to Berkeley, Calif., where he remained for the next twelve years. From 1900 to 1905, he was Superintendent for Rudolph Axman and Robert Wakefield, Contractors, in the removal of Skagg, Arch, and Blos-

* Memoir prepared by Frank Adams and Paul Bailey, Members, Am. Soc. C. E.

som Rocks to a depth of 30 ft. from San Francisco Bay. In 1905, he was elected City Engineer of Berkeley, serving in that capacity until 1909, when he became Commissioner of Public Works at the time Berkeley adopted the Commission Form of Government.

In 1911 he returned to private engineering practice, continuing in this until his appointment by Governor Hiram Johnson, on February 13, 1912, to the office of State Engineer of California. From that appointment dates his largest engineering service. Through three successive gubernatorial administrations and a service exceeding fourteen years Mr. McClure continued as State Engineer of California. From July 30, 1921, until his death, in addition to holding the position of State Engineer, he was Chief of the Division of Engineering and Irrigation of the State Department of Public Works, and from August 17, 1923, Director of that Department.

At the time of Mr. McClure's appointment by Governor Johnson in 1912 the jurisdiction of the State Engineer's Office related principally to work in and about State institutions, with minor enterprises in road construction and rectification of river channels. During the period in which he was State Engineer, however, the activities of the office underwent a great expansion. State supervision of irrigation and reclamation development and of dam construction originated and grew to large proportions. Engineering investigational work also became an important function of the office.

Between 1911 and 1915 the California Irrigation District Act was perfected, through many amendments, to the point where it became practical to proceed with substantial irrigation development. In 1913, the California Irrigation District Bond Commission (now the California Bond Certification Commission), composed of the State Engineer, Attorney General, and Superintendent of Banks, was created by the Legislature. Through the separate jurisdiction of the State Engineer's Office under the California Irrigation District Act and the jurisdiction of the Bond Commission, the State, since 1912, has supervised one of the greatest expansions in agriculture through irrigation that has occurred in any locality in the United States. This expansion has involved the organization of about 100 irrigation districts and the issuance of more than $120 000 000 of irrigation district bonds for construction work. The organization of these districts, and the development of their irrigation systems all passed under Mr. McClure's official review. The total area in these districts is 3 500 000 acres. This is a greater area than is at present (1926) irrigable under all the Federal irrigation projects combined.

The assemblage of the large volume of capital at low interest rates necessary to construct the works for irrigating this vast area was made possible through the successful administration of State supervision of irrigation projects. This successful administration has made irrigation securities readily purchasable by the buying public. Almost without exception, these entire areas have met their financial obligations as they became due.

This great and rapid expansion of the irrigated areas in California advanced conditions to the point where organization of large areas became necessary under conditions not contemplated by the California Irrigation District Act.

Accordingly, the California Water Storage District Act was passed by the Legislature of 1921 and the California Water Conservation District Act in 1923. Under the supervision of the State Engineer as provided in these Acts, districts of about 2 300 000 acres have organized to irrigate new lands and to improve the water supply of other lands already under irrigation.

The great increase in productivity resulting from the successful irrigation of these large areas has enabled California to take a ranking position among the States of the Union in agricultural production. Without irrigation California would have reached its maximum production as long as forty years ago. At that time the greatest area profitable to cultivate by dry-farming methods had been brought into use. Through the last forty years, by the introduction of irrigation and without increasing the total area under cultivation, the value of agricultural production in the State has increased manyfold.

In order that progress might continue uninterruptedly and that agriculture and other industries dependent on water might further contribute to the prosperity of the State, engineering investigational work concerning available water supplies and their most advantageous utilization became one of the most important functions of the State Engineer's Office during the later years of Mr. McClure's administration. This work produced the reports on the water resources of the State presented to the Legislatures of 1923 and 1925. These reports contained the first complete inventories of the waters of California and of the future needs for water in order to accommodate the full development of the State's resources.

The culminating report to this work was under preparation at the time of Mr. McClure's death. This report presented to the Legislature of 1927 a plan for co-ordinating the development of the State's waters for all purposes, and for equalizing the naturally unequal distribution of such waters to the end that all sections of the State may receive their full quota.

Parallel with this immense program of irrigation development, the reclamation of overflow lands in the Sacramento and San Joaquin Valleys has also progressed rapidly under legislation passed in 1911. The State Engineer's Office has participated with the State Reclamation Board and the California Débris Commission, as provided in the legislation for the State supervision of these projects. All this work stands as a monument to the administration of Mr. McClure.

Possibly one of the most important engineering services rendered by him was as California member of the Colorado River Commission, which was organized on January 27, 1922, under the Chairmanship of the Secretary of Commerce, Herbert Hoover, Hon. M. Am. Soc. C. E., and which practically concluded its work with the signing of the Colorado River Compact at Santa Fé, N. Mex., on November 24 of the same year.

In a letter to the writers, Secretary Hoover states that Mr. McClure's "unceasing devotion in the interests of the State of California was accompanied by a breadth of vision as to national interests which marked his character at all times," and "his engineering knowledge and familiarity with all the facts bearing on the most difficult questions and his fine personality made

him one of the most important members of the Commission." The controversies that followed the signing of the Compact at Santa Fé were a constant source of concern to Mr. McClure, and to his death he continued to lend his influence toward a fair and equitable solution of what he recognized to be a problem of overshadowing importance to California and the entire Southwest.

If there is one characteristic that stands out above others in Mr. McClure's administration of the important office he occupied from 1912 to his death, it was his desire to perform his duties fully and faithfully and to act with absolute fairness and justice. He knew no greater professional pleasure than to help forward an enterprise he deemed to be worthy, and it was always with genuine regret that he withheld approval from one, sincerely undertaken, which he considered economically or physically unjustified. Although never seeking official authority, he did not hesitate to exercise that given to him by law, and would brook no subterfuge or dishonesty. His stand on matters before him was always based on his own ideas of right. It was his nature to side with the smaller and weaker if convinced they were honest in their endeavors, and because of his total lack of self-interest he was always able to inspire loyalty and the utmost effort among those working under him. The same fine qualities that endeared him to his family and those of his organization, endeared him also to all who came into friendly contact with him.

Mr. McClure was devoted to his family. He was first married at Candor, Pa., on August 17, 1882, to Sarah Hunter McCalmont, whom he met while attending the Normal School at California. There were four daughters of this marriage—Mrs. J. W. Mahoney, of Berkeley, Calif., Mrs. G. A. Robinson, of Merced, Calif., Miss Katherine McClure, of Sacramento, Calif., and Clara Clark McClure, who died in infancy. Following the death of his first wife in 1915, he was married on October 9, 1916, to Margaret Alter, a friend of his childhood, who survives him.

Not only was Mr. McClure devoted to his family, but he entertained the deepest feeling for his fellow men. Almost invariably he was an officer of his church, and both in Berkeley and in Sacramento, where he resided at the time of his death, he was also an officer in the Young Men's Christian Association. All worthy causes appealed to his kindly sympathy, and to these he was a consistent and conscientious tither.

Mr. McClure was elected a Member of the American Society of Civil Engineers on November 3, 1886, but resigned on December 31, 1894. He was elected a Member a second time on December 14, 1925.

LEONARD METCALF, M. Am. Soc. C. E.*

DIED JANUARY 29, 1926.

Leonard Metcalf was born at Galveston, Tex., on August 26, 1870, the son of Joseph Houghton and Emma Augusta (Leonard) Metcalf. For the greater part of his life his home was in Concord, Mass. He prepared for college in the Concord High School. In 1892 he was graduated from the Civil Engineering Course at the Massachusetts Institute of Technology. During his student days he was President of the Civil Engineering Society at the Institute. He was also a member of the Delta Kappa Epsilon Fraternity.

Mr. Metcalf was employed for the three years immediately following his graduation by Wheeler and Parks, of Boston, Mass., Civil Engineers and Operators of Water Companies. For the next two years, he was Professor of Mathematics and Engineering at the Massachusetts Agricultural College, and, in 1897, he began practice in his own name as a civil engineer in Boston. In 1907 Mr. Metcalf, with Harrison P. Eddy, M. Am. Soc. C. E., formed the partnership of Metcalf and Eddy to practice hydraulic and sanitary engineering, which association he maintained for the remainder of his life.

In the field of municipal water-works he achieved notable success. His practice involved the design, construction, and operation of water supply systems in many parts of the country. For years he served as manager of public service corporations operating in the fields of water supply and electric light and power plants. He was particularly interested in the financial problems of water-works properties and was called upon almost constantly to advise owners relative to the betterment of economic conditions. He was insistent upon sound policies which would bring about, not only suitable compensation to owners, but adequate service to consumers.

In the field of valuation and rate-making for public utilities his accomplishments were of outstanding importance. He was prominent among a small group of engineers who by their honesty, judgment, and technical knowledge have greatly influenced present-day conceptions of the factors entering into the fair value of public utility systems. Going value, depreciation, and price trends were matters upon which Mr. Metcalf brought to bear the logic of his clear mind.

Recognition of his fair-mindedness is shown by the following extract from the report of the Hon. H. M. Wright, Master in the case of the Spring Valley Water Company *vs.* San Francisco, 1917:

"Mr. Metcalf had charge of the preparation of plaintiff's case on its technical and general aspects, so far as completeness and orderliness of presentation were concerned. It might be expected that in such an employment he might imbibe some of the partisan character of the advocate, but happily his evidence, and that of Mr. Ellis, who performed like service for the city, was quite devoid of any appearance of bias."

* Memoir prepared by Harrison P. Eddy, Allen Hazen, and Charles W. Sherman, Members, Am. Soc. C. E.

Another testimonial of this nature is contained in a letter from the Hon. Charles E. Gurney, Chairman of the Public Utilities Commission of Maine, to one of Mr. Metcalf's partners, reading, in part, as follows:

"Among the emoluments of my present position, more attractive to me than the financial return, are some of the men I have met, whose lives and achievements inspired my admiration and an endeavor to do my best work each day. None of them has impressed me more deeply for qualities of character, a manifest integrity and a steadying earnestness of purpose than Mr. Leonard Metcalf of your firm. I always felt as he testified before this Commission that there was no danger of his stultifying himself by making any statement in which he himself did not place implicit confidence. I felt that he would not exaggerate wantonly and that I could depend upon what he told me. I do not mean by this that I closed my own mind and accepted his word as infallible, but I do mean I felt that so far as human error honestly made might be excluded, Mr. Metcalf's word might be accepted and that he would not sell his opinion to the highest bidder."

As his reputation in this field became established, Mr. Metcalf's advice in cases involving water-works valuation and rates was frequently sought. Among the most important of such cases were those of the water-works at San Francisco, Calif., Denver, Colo., Des Moines, Iowa, Indianapolis, Ind., Paterson, N. J., and Utica, N. Y. Worthy of special mention is the water-rate case of the Pennsylvania Water Company of Wilkinsburg, Pa. In this case he made a study of the basis of fixing rates for fire hydrant service which has been accepted generally as the most significant and authoritative discussion of the matter thus far presented.

As a result of his extensive experience and sound judgment in the financial aspects of water supply, Mr. Metcalf was frequently called to advise capitalists in matters relating to the purchase of water supply properties. Among his clients may be mentioned, the original Boston Finance Commission (1907-08); Dayton, Ohio; East Chicago and Indiana Harbor Water Company; East Providence Water Company; Fitchburg, Mass.; Gloversville, N. Y.; Kingston, N. Y.; Macon, Ga.; Milford, Mass., Water Company; Pennichuck Water-Works, Nashua, N. H.; Nashville, Tenn.; Pawtucket, R. I.; Peoria, Ill., Water Company; Plattsburgh, N. Y.; Portland Water District, Maine; Kennebec Water District, Maine; Rumford and Mexico Water District, Maine; Dover-Foxcroft Water District, Maine; San Antonio Water Supply Company; Spring Brook Water Supply Company (Wilkes-Barre, Pa.); United Fruit Company; Penobscot County Water Company, Maine; Tampa, Fla., Water Company; Woburn, Mass.; Syracuse, N. Y.; San José, Costa Rica; and many other municipalities and corporations. During 1925, he was Consulting Engineer to the Metropolitan Water Supply Investigating Commission of Massachusetts.

Few engineers have given themselves so unstintedly to their profession as did Leonard Metcalf. He was a member of many engineering societies and served on numerous committees. He was a member of the American Society of Mechanical Engineers; of the Boston Society of Civil Engineers, of which he was President in 1919; of the American Water Works Association (President, 1916-17); and of the New England Water Works Association, being

elected President in 1915 and later an Honorary Member. He was active in the formation of the Affiliated Technical Societies of Boston directly after the World War.

His numerous writings on professional subjects, published in the *Journals* of the American and New England Water Works Associations, in the *Transactions* of the American Society of Civil Engineers, and in the recently issued "Manual of American Water-Works Practice", illustrate his unselfish desire to share with other engineers the fruits of his study and experience. Among his important contributions to technical societies are the following:

For the American Society of Civil Engineers: "The Antecedents of the Septic Tank";* "The Groined Arch as a Covering for Reservoirs and Sand Filters: Its Strength and Volume";† "Water-Works Valuation and Fair Rates, in the Light of the Maine Supreme Court Decisions in the Waterville and Brunswick Cases";‡ "The Going Value of Water-Works"§ (co-author with John W. Alvord, M. Am. Soc. C. E.); and "Final Report of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities".‖ (Mr. Metcalf was Secretary of the Committee.)

For the American Water Works Association: Parts of several chapters in the "Manual of American Water Works Practice" (1925); "Some Fundamental Considerations in the Determination of the Reasonable Return for Public Fire Hydrant Service" (co-author with the late Emil Kuichling and W. C. Hawley, Members, Am. Soc. C. E.); "Some Practical Checks on Water Works Depreciation Estimates"; "Experiences with Ice in Standpipes"; "The War Burdens of Water-Works"; and "The Improved Financial Condition of Water Works".

For the New England Water Works Association: "Depreciation in Water Works"; "Echo Lake Dam at Milford"; "Wrought-Iron Cement-Lined Pipe"; "Data on Awards for Water and Water Power Diversion" (report of a Committee of which Mr. Metcalf was Secretary).

He was a joint author with Harrison P. Eddy, M. Am. Soc. C. E., of the three-volume treatise on "American Sewerage Practice" and of the one-volume abridged edition on "Sewerage and Sewage Disposal" which is now used in approximately sixty universities and colleges in the United States.

Mr. Metcalf was a loyal and active Alumnus of the Massachusetts Institute of Technology. He was President of the Alumni Association in 1920 and was a Term Member of the Corporation of the Institute at the time of his death. Evidence of the high regard in which he was held by the Institute is afforded by the fact that he was offered its Presidency as a successor to the late Dr. Richard C. Maclaurin.

Not only to professional and educational matters did he give of his time and energy, but also to public affairs. He was a member of the Boston

* *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), p. 456.
† *Loc. cit.*, Vol. XLIII (1900), p. 37.
‡ *Loc. cit.*, Vol. LXIV (1909), p. 1.
§ *Loc. cit.*, Vol. LXXIII (1911), p. 326.
‖ *Loc. cit.*, Vol. LXXXI (1917), p. 1311.

Chamber of Commerce and served on its committees. In Concord, he was for several years Trustee of the Public Library, and from 1915 to his death, he was a member of the Board of Water and Sewer Commissioners, having served as Chairman from 1917.

When the United States entered the war with Germany and the Central Powers in 1917, Mr. Metcalf was appointed a member of the Sub-Committee on Emergency Construction of Buildings and Engineering Structures, under the National Council of Defense. This Committee assisted Gen. I. W. Littell in establishing the Construction Division, selecting personnel, preparing forms of contract, providing lists of contractors, selecting cantonment sites, and awarding contracts for the National Army cantonments erected during 1917. The value of the services rendered by these civilian engineers during the early days of the war is inestimable. Mr. Metcalf did his full share in this important work.

His standards of professional honor and ethics were high. His energy and accomplishments were remarkable. He attacked each new problem with keen intellectual zest, clear vision, and sound judgment. He was always interested in the welfare of his associates and was always ready to counsel young engineers. His enthusiasm, optimism, and vigor impressed all with whom he came in contact. These qualities brought him, not only respect, but real affection. Although leading an extremely busy professional life, he found time for recreation. He was fond of Nature, particularly of the mountains of California and Colorado. In the latter State he had a summer home in which he took great pride and pleasure. Walking, fishing, and riding were outdoor pastimes which strongly appealed to him. He was appreciative of art and literature. With his vigorous personality there was combined the charm of a broadly cultured mind.

He was a member of the Union Club of Boston, the Engineers' Clubs of Boston and New York, the University Club of Chicago, and the Social Circle of Concord.

Mr. Metcalf never married. He was devoted to his mother during her life and to her memory after her death. His loyalty to his two sisters and to more distant relatives was one of his marked characteristics.

During the last months of his life, suffering from an incurable disease, he faced the pain and the inevitable end without complaint and with unfaltering courage.

Mr. Metcalf was elected an Associate Member of the American Society of Civil Engineers on January 5, 1898, and a Member on September 2, 1903. He served as a Director of the Society in 1913 and 1914 and as Vice-President in 1919 and 1920.

MARSHALL MORRIS, M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1925.

Marshall Morris was born in Louisville, Ky., on May 30, 1846. He was the youngest son of Joseph Saunders and Eliza (Morton) Morris and a grandson, on the maternal side, of John Hite and Sarah (Price) Morton, of Lexington, Ky. His father was the seventh generation of the Morris family in this country.

Thomas Morris, an ancestor, married Abigail Marshall, granddaughter of Christopher Marshall. The Morris and Marshall families were of Philadelphia, Pa., both families having come from England in early Colonial days, the Marshalls to escape Quaker persecutions. Christopher Marshall, with a number of others, was excommunicated by his Church for assisting the Revolutionary cause. These men formed the Society of Free Quakers, in which the older men gave their time and means to aid the Revolution and the younger ones joined the Army.

In 1863, at the age of seventeen, Mr. Morris was graduated from the University of Louisville, receiving the degree of Bachelor of Science.

Immediately after graduation he joined a surveying party on railroad location work in Eastern Kentucky and was employed on that work for several years, being Chief of Party on the survey of the Louisville and Cincinnati Railroad in 1866.

In 1871 and 1872 he played a prominent part in the building of the Elizabethtown and Paducah Railroad under Mr. George McLeod. At the close of that assignment he went to Texas as Chief of Party to make the early survey for a part of the Texas and Pacific Railroad from Fort Worth, west, during which the surveyors were much harassed by Indian interference.

Mr. Morris then opened and operated the Jefferson Coal Mines, at Jefferson, Ala., but remained there only a few years before returning to railroad construction work, including bridges, tunnels, etc. Later, he became Chief Engineer for the old Louisville, New Albany and Corrydon Railroad (now the Monon Division of the Chicago, Indianapolis, and Louisville Railroad). This position he resigned to enter into partnership with Mr. John Serpelle, as contractors on railroad construction work.

About 1888 he located the rock asphalt deposit in Breckenridge County, Kentucky, and organized the Breckinridge Asphalt Company, serving as its President. This Company laid many streets in the larger cities, Louisville, Buffalo, N. Y., etc., placing more than twelve miles of pavement in Buffalo alone.

In 1908 Mr. Morris was appointed Building Inspector for the City of Louisville, drafting the Building Code which was put into effect in 1909. From this time he was retired from active field work, doing only such consulting work as his then failing eyesight would permit.

* Memoir prepared by J. M. Johnson, M. Am. Soc. C. E.

In 1876 he was married to Tenie Witherspoon, of Longview, Tex., who died in 1904.

His death occurred on September 29, 1925, at the residence of his daughter, Mrs. George A. Haynes, at Bedford, Ind., and he was buried in Cave Hill Cemetery, Louisville, Ky. He is survived by his daughter, Mrs. Haynes, and two sons, Marshall Morris, M. Am. Soc. C. E., of Austin, Tex., and John Ford Morris, of Louisville.

Mr. Morris was a member of the Engineers and Architects Club of Louisville. He was also a Charter Member of the Louisville Club, acting as its first Secretary, and bequeathing to it his valuable collection of the *Transactions* of the Society.

Mr. Morris was elected a Member of the American Society of Civil Engineers on March 5, 1873.

EMIL NEWMAN, M. Am. Soc. C. E.*

DIED OCTOBER 3, 1926.

Emil Newman was born at Frotuna, in the Province of Nerike, Sweden, on February 11, 1854. His father was Gustav Nymann (the Swedish spelling of Newman) and his mother, Geana Fredericka (Hesslius) Nymann. The family is noted in the religious and professional history of Sweden. One of Mr. Newman's grandfathers was a "Magister Companius" (Professor), and was one of the early settlers in Pennsylvania. There he founded and built, in 1646, the Old Swedes' Church, one of the historic landmarks of Philadelphia. This famous old house of worship still stands and is used regularly.

Mr. Newman received his education at the Polytechnic Institute of Orebro, Sweden, from which he was graduated with high honors in 1873. In the same year he came to the United States in order to follow his ambitions in the engineering field. His professional career may be divided into two parts: The years that he spent in railroad service, and the later years when he was engaged in hydro-electric engineering.

From 1873 to 1880, he was connected with the Engineering Corps of the Wisconsin Valley Railroad Company, in the capacity of Chainman, Transitman, and, later, in charge of a Division, consisting of 7½ miles of main line, and several miles of sidings. He then became Assistant Engineer, and was placed in charge of maintenance of track, bridges, and building, and, later, as Locating and Construction Engineer, he was in charge of line extensions.

From 1880 to 1883, Mr. Newman was with the Chicago, Milwaukee, and St. Paul Railroad Company as Assistant Engineer in charge of railroad extensions and Engineer in charge of road maintenance and reconstruction on the Wisconsin Valley Division. In 1883 and 1884, he was General Roadmaster for the Canadian Pacific Railroad Company, in charge of track from Winnipeg, Man., to Port Arthur, Ont., and from Winnipeg to Emerson, Man. During this period, he was in charge of the reconstruction of 300 miles of line from Rat Portage to Port Arthur, and also the building of grain elevator foundations and docks at Port Arthur. From 1884 to 1886, he was Resident Engineer of the Chicago, Burlington, and Northern Railway Company at La Crosse, Wis.

He continued his railroad work until 1887, when a rapid advancement was interrupted by the necessity of moving to California on account of the health of his only daughter, Iris. He went directly to Riverside, Calif., and from 1887 to 1888 he had charge of the construction of the main sewer and main domestic water supply of that city.

From 1888 to 1896, Mr. Newman was employed as Manager of the Pioneer Bank and Pioneer Land Company at Porterville, Calif., during which time he built sewage works, water-works, and an electric light plant, and was actively engaged in various phases of municipal and hydraulic engineering.

* Memoir prepared by Harold K. Fox and George L. Swendsen, Members, Am. Soc. C. E.

He was also engaged, for a short time, in prospecting for gold mines in California, but evidently found this field of endeavor to be more romantic than profitable for, in 1897, he again turned to railroad engineering, and until 1900 was Water Engineer for the Atchison, Topeka and Santa Fé Coast Lines, and from 1900 to 1903, Construction and Locating Engineer for the Santa Fé in Texas.

Mr. Newman entered the employ of the San Joaquin Light and Power Corporation in 1907. Previous to that time he had done considerable hydro-electric work for the Pacific Light and Power Corporation, having been in 1903 Assistant Engineer for this Company in full charge of the construction of a 1 500-h.p. plant in Santa Ana Canyon, California. His first assignment with the San Joaquin Light and Power Corporation was a survey of the Yosemite Valley Railroad from Fresno to Wawona, Calif. After its completion, he took charge of additions and reinforcements to the rock-fill dam on the Crane Valley System.

In 1912 and 1913, he had charge of building part of the road into what is now known as the Big Creek Project of the Southern California Edison Company, after which he was connected with the engineering on the Tule River Project. He then returned to the Crane Valley System, where he built the No. 2 Conduit and No. 2 Power-House. This conduit consists of lined ditch, flumes, and tunnels. The power-house had a capacity of about 1 000 kw. and was of the latest type at that time. He then built an automatic plant, known as the 1-A Power-House, which utilizes a head of about 30 ft. between the conduit and forebay of the No. 1 Plant.

In April, 1921, Mr. Newman made a trip to the Scandinavian countries, spending several months visiting notable hydro-electric plants in Sweden and Norway, and making comparative studies between foreign methods and those of the United States. While in Sweden, in addition to the pleasure of being with his two sisters whom he had not seen in many years, he also renewed the acquaintance of three old classmates from a class of eleven that was graduated forty-eight years prior to their reunion.

In 1923, he was appointed Consulting Engineer for the San Joaquin Light and Power Corporation and served in this capacity until his death.

On May 10, 1878, Mr. Newman was married to Lula F. Woodley, who died on January 8, 1921. He is survived by his daughter, Iris, of Fresno, Calif., and by two sisters, Maya and Augusta Nymann, who reside in Sweden.

In the passing of Emil Newman, the Engineering Profession has sustained a great loss. His wonderful vision, high ideals, absolute integrity and loyalty, his excellent judgment and fair-mindedness, together with a deep sense of humor, are among the many admirable characteristics which engendered the respect and admiration of an exceptionally large circle of friends.

He was a member of the Fresno Chapter of the American Association of Engineers, and this organization is completing plans for the erection of a bronze tablet at some point on the San Joaquin River, in the mountains he knew so well, to commemorate his life and achievements as an engineer.

Mr. Newman was elected a Member of the American Society of Civil Engineers on January 4, 1905.

ASHBEL EDWARD OLMSTED, M. Am. Soc. C. E.*

Died August 15, 1926.

Ashbel Edward Olmsted, the son of Ashbel and Emeline (Stanley) Olmsted, was born in East Hartford, Conn., on December 21, 1845. He was descended from James Olmsted, one of the first settlers of Hartford, Conn., who came through the "wilderness" with Parson Thomas Hooker from Cambridge, Mass., in 1636.

At the age of thirteen Mr. Olmsted entered the Hartford High School where he was a student for two years, leaving before his graduation on account of weak eyes. He then assisted his elder brother on the farm, but continued the study of mathematics at home. After an extended visit with relatives in Iowa he returned with the settled idea of becoming a civil engineer. In his twenty-first year he expected to enter Yale College, but on account of a dangerous gunshot wound received while hunting in Iowa, he was obliged to give up his college education. Nothing daunted, however, he continued his studies at home with the help of Hiram Loomis, an engineer of Hartford, and in 1868 secured his first position on a division of the Troy and Greenfield Railroad east of the Hoosac Tunnel, under the late John Waldo Ellis, M. Am. Soc. C. E., Division Engineer.

In 1869, when the Connecticut Valley Railroad surveys were started, Mr. Olmsted held for a short time the position of Flagman, but was soon advanced to that of Rodman, then, as the Transitman failed to appear when the engineering party was made up, he took levels and did transit work until, in 1870, he was placed in charge of a Division from Deep River, Conn., to Long Island Sound, which position he held for three years.

From 1873 to 1879 he did various kinds of engineering work in different parts of Connecticut and in his home town, but again turned to railroad construction in the summer of 1879 when he became Assistant Engineer on the New York and New England Railroad under L. B. Bidwell, Chief Engineer. A year later Mr. Olmsted was placed in charge of the new line in Putnam County, New York, with headquarters at Towners, and, later, had charge of the terminal work at Fishkill-on-the-Hudson.

His next position was on the New York, West Shore, and Buffalo Railroad, where he had charge of twenty-one miles under Mr. Thomas Spencer. In January, 1883, the late Elmer L. Corthell, Past-President, Am. Soc. C. E., advised him to go to Syracuse, N. Y., to interview the members of the firm of Ryan and McDonald (John B. McDonald), which was starting extensive work on the West Shore Railroad. An agreement was made between them, and for a few weeks Mr. Olmsted was with Mr. McDonald at Syracuse, later going to Lyons, N. Y., where thirty-four miles of railroad was being built under Mr. Ryan.

* Memoir prepared by William J. Boucher, M. Am. Soc. C. E., from data furnished by relatives.

In the fall of 1883, the West Shore Railroad failed, but Mr. Olmsted remained to settle the firm's affairs until the spring of 1884, when Mr. McDonald assigned him to work in connection with the extension of the Baltimore and Ohio Railroad from Baltimore, Md., to Philadelphia, Pa. He was located at Chester, Pa., for two years. While engaged on this work he first met Colonel (later, General) Henry T. Douglas, Chief Engineer of the Baltimore and Ohio Railroad. Thus began an acquaintance and friendship that lasted until the death of General Douglas in July, 1926.

One day in the fall of 1885, when Mr. McDonald asked Mr. Olmsted how soon he could go to Chicago, Ill., he answered "to-night", and by midnight he was on his way to look over eighty miles of heavy work on the Illinois Central Railroad from Elgin to Freeport, Ill. Soon afterward, the firm secured the work and Mr. Olmsted went to Rockford, Ill., where he remained for a year. Later, the firm built fifty-seven additional miles from Freeport, Ill., to Dodgeville, Wis. It was a rush job, the contractor having agreed to build it in four months.

In the spring of 1888, Mr. Olmsted went to Seattle, Wash., a very little known city at that time, to look over work on the Seattle and Eastern Railroad in the Cascade Mountains. In August of the same year, he and his family moved to the Pacific Coast where they lived at one of the largest hop ranches in the country at Snoqualmie, Wash., where he made his headquarters. He was there only a short time as the firm found financial conditions such as not to warrant continuing the construction of the line. He went to Tacoma, Wash., for a short time, but as conditions in the Pacific Northwest were not favorable to a civil engineer seeking employment, he returned East.

During the summer of 1889 he was employed in the coal fields of West Virginia and, later, was with the Philadelphia and Reading Railroad Company, in charge of the masonry work for a large bridge over the Schuylkill River, with headquarters in Norristown, Pa. His last work with the firm of Ryan and McDonald began in 1890 and lasted two years, when he was engaged on the construction of the Western Maryland Railroad, and resided at Williamsport (near Hagerstown), Md.

In 1892 Mr. Olmsted went to Panama in connection with a manganese mine, but remained there only a few months as the climate was too great a strain on his health. In 1893, Mr. McLeod, President of the New York and New England Railroad Company, was trying to get an entrance for that railroad into New York, N. Y., and proposed building about fifty miles of road from Brewster, N. Y., to New York City. It was to be known as the New York, New England, and Northern Railroad. Mr. Olmsted was appointed Chief Engineer, with headquarters in White Plains, N. Y., and had two or three engineering parties in the field, but in December of that year the railroad went into the hands of a receiver and the work came to an end.

Mr. Olmsted's next work was at the Croton Dam, where he endeavored to construct a railroad line from the Town of Croton to the dam. This,

however, was not possible as there was no way to force settlements of land damages, but he was able to build a short road from a stone quarry to the work. In the summer of 1894 he located a railroad to a phospate mine in Tennessee.

The years between 1895 and 1899 were spent by Mr. Olmsted as Consulting Engineer for the Central New England Railroad Company and at the Jerome Park Reservoir, which Mr. McDonald was building for the City of New York. A part of the work was the construction of a railroad line from Jerome Park to near Hunts Point, Borough of The Bronx, which was used for transporting the excavated material and disposing of it on the large area of marsh land in that vicinity.

In the fall of 1899, the first subway was offered for bidding by the Rapid Transit Commission of New York City. Mr. McDonald planned to bid on the work and to Mr. Olmsted he entrusted the investigations and studies of this stupendous project. It was indeed a new subject, for at that time only meager knowledge existed of the subsurface conditions of New York's streets. It is true that borings had been made to explore for earth or rock conditions, but the real problem was the location of water and gas pipes, sewers, and electric conduits, the exact data for which were lacking with City departments and utility companies, the drawings accompanying the contract denying responsibility for the accuracy of the information given. Mr. Olmsted spent a month on the study and at its conclusion Mr. McDonald put in the now famous bid for the entire work, the construction of the subway in Manhattan, viaducts in The Bronx, twenty-one miles in all, including the Harlem River Tunnel and Ship Canal Bridge, and the reconstruction, relocation, or restoration of all subsurface structures encountered, for the lump sum of $35 000 000. There was only one other bid, which was higher.

No contract had ever been offered or awarded for such a large amount of money and certain it is, that none had ever been considered which involved such a variety of work to be done under such uncertain conditions. No praise can be too great for the painstaking effort, for the skill exhibited, and for the keen insight which was necessary in order to grasp the problem and form an opinion of the costs and methods of accomplishing the great task. This Mr. Olmsted did.

After the award of the contract, he was offered the position of Chief Engineer, but felt he should not accept it on account of his health. He was, therefore, appointed Division Engineer, a position which he filled with the Rapid Transit Subway Construction Company (the company organized to take over the McDonald contract) and with the Interborough Rapid Transit Company, until his retirement in 1918. In this position he was actually in charge, under the late S. L. F. Deyo, M. Am. Soc. C. E., Chief Engineer, of the construction work, visiting the various sections, at the beginning, almost every day and, at the office, checking quantities and going over the estimates prepared by the Resident Engineers. Later, Contract No. 2, for the extension of the subway from Ann Street, Manhattan, under the East

River to Atlantic Avenue, Brooklyn, was also constructed by the same Company, and Mr. Olmsted performed similar duties as on Contract No. 1. The responsibility for the accuracy of the estimates, and consequently for the financial side of the work, was his. His name appears on the memorial tablets in both the City Hall Station, Manhattan, and Borough Hall Station, Brooklyn.

After his retirement in 1918, Mr. Olmsted returned to his boyhood home at East Hartford, where he read, studied, and worked in his garden until six months before his death, which occurred on August 15, 1926, after a lingering illness. He was married to Emma Leland Strickland, of Essex, Conn., on August 5, 1872, who died in 1903. Mr. Olmsted is survived by an adopted daughter, Ella S. Olmsted.

Mr. Olmsted was of a most agreeable, courteous, and kindly disposition. Those in trouble who applied to him, went away either with substantial assistance or good advice. Painstaking, methodical, careful, and accurate in his work, he was a loyal assistant to all his employers, and at the same time absolutely honest in seeing both sides of the proposition and coming to a decision only after weighing all the evidence impartially.

He was a life-long member of the Congregational Church, a member of the New York Academy of Sciences, and also a member of the Republican Club of Hartford.

Mr. Olmsted was elected a Member of the American Society of Civil Engineers on May 6, 1903.

CHAUNCY RUSCH PERRY, M. Am. Soc. C. E.*

DIED JUNE 24, 1926.

Chauncy Rusch Perry was born at Waltham, Mass., on May 6, 1872. He was the son of the Rev. John Philander and Emma (Rusch) Perry. His boyhood was spent mostly in New Ipswich, N. H., to which town his parents removed while he was a child. His preliminary education was obtained in the public schools of New Ipswich and the New Ipswich Academy of which institution he later became a Trustee.

In June, 1890, Mr. Perry entered the Engineering Department of the Boston Bridge Works where he remained for two years, acquiring experience in drafting and fitting himself for higher education. In September, 1892, he entered the Lawrence Scientific School and was graduated therefrom in June, 1895, with the degree of Bachelor of Science.

For the next three years Mr. Perry was employed by the Boston Transit Commission under Howard A. Carson, Hon. M. Am. Soc. C. E., first as a Draftsman and later as Assistant Engineer on the design of the steel framework of the Boston Subway. After an interval of eight years, during which he was with J. R. Worcester, M. Am. Soc. C. E., Consulting Engineer, and with the Boston Bridge Works, designing and estimating, he returned to the Transit Commission, by which he was employed as Assistant Engineer until 1910. During this time he was engaged in the design of many complicated parts of the Boston subways, tunnels, and stations, dealing not only with structural steel framework, tunnel shields, etc., but also with reinforced concrete sections.

For the greater part of the last sixteen years of his life Mr. Perry was connected with J. R. Worcester and Company, although at various times he undertook independent engagements. The latter included the design of the reinforced concrete of the Germanic Museum of Harvard University and of the Morgan Memorial in Boston; the design of the steelwork of the Providence Armory; a study of standard library stacks for the Library Bureau; buildings and crane tracks for the Hunt-Spiller Manufacturing Corporation, South Boston; the Squantum Destroyer Plant; Alameda Shipyards and Hog Island Plant for the United States Navy; Fore River Battleship Yard; and an exhaustive examination of bridges of the United Fruit Company in Costa Rica. He gave much thought to the design of tunnel shields, which were used in the construction of various tunnels.

Mr. Perry was married on June 21, 1896, to Helen Tuttle, who, with a daughter and three sons, survives him. He had few interests outside his home life and his work, although he was a violinist of ability and devoted what spare time he had to music. He was a persistent student and investigator, and delighted in solving complicated problems in his line of work. His clear reasoning and balanced judgment were relied on by his employers and associates when difficult questions were involved. He was a faithful

* Memoir prepared by J. R. Worcester, M. Am. Soc. C. E.

member of the New Jerusalem Church, and without ostentation endeavored to carry into actual life the teachings of his religion.

In disposition and bearing Mr. Perry was modest, deferential, and tolerant of the shortcomings in others, but firm in facing unpleasant situations, stoutly maintaining his side in an argument when convinced of its correctness. His nature was optimistic, always looking for the cheerful features in every occurrence. His mind was ever active, acquiring information of all kinds, which he was ready to share with others on every suitable occasion. Although connected with the Society for twenty-one years, he did not attend its meetings or take part in the discussions. He did, however, value highly its publications and made full use of the information derived therefrom.

Early in 1926 he suffered a severe attack of influenza from which he did not fully recover his strength. He kept at his work, however, until the day of his death, which occurred without a moment's warning on June 24, 1926.

Mr. Perry was elected an Associate Member of the American Society of Civil Engineers, on February 1, 1905, and a Member on June 1, 1909.

THOMAS ELMER PHIPPS, M. Am. Soc. C. E.*

DIED FEBRUARY 22, 1926.

Thomas Elmer Phipps was born near Janesville, Ill., on May 18, 1879, the son of Napoleon and Hannah Phipps. After completing his studies in the public schools Mr. Phipps taught school in Coles and Cumberland Counties, Illinois. In 1910 at Valparaiso, Ind., he received a Bachelor of Science degree from Valparaiso College, and after his graduation, became Principal of the Public Schools at Lerna, Ill. In 1903 Mr. Phipps entered the University of Illinois and completed the four-year course in 1906, graduating with high honors.

During the vacation periods he was employed as Instrumentman on the Cairo Division of the Cleveland, Cincinnati, Chicago, and St. Louis Railway. After graduating from the University until February, 1907, he was Resident Engineer of the Division, in charge of forty-two miles of reconstruction work.

During the summer of 1907, Mr. Phipps became Superintendent of the Walsh Construction Company, in charge of steam-shovel crews, reconstructing and double-tracking the main line of the Cleveland, Cincinnati, Chicago, and St. Louis Railway between Terre Haute and Indianapolis, Ind. On the completion of this work he went to Seattle, Wash., where he entered the City Engineer's Office, remaining until April, 1908.

Mr. Phipps then entered the service of the Chicago, Milwaukee, and St. Paul Railway Company and was in its Engineering Department until October, 1914. During this time he was for two years Chief Draftsman in the Division Office at Ellensburg, Wash., and one year in charge of the construction of fourteen miles of the mountain end of the Everett Line. From November, 1911, to October, 1914, he served as Assistant Engineer in charge of locating parties on the Olympic Peninsula; of a survey of the Bellingham Northern Railway; of investigations and locations of transfer landings on Puget Sound; of building landings and terminal tracks in Bellingham; and of maintenance work.

In October, 1914, he became Assistant Engineer for the Public Service Commission of the State of Washington and Chief Engineer of the Commission in February, 1915. This Commission has jurisdiction over the valuation, rates, and service regulations of all public utilities in the State of Washington. Among the more notable cases in which Mr. Phipps as Chief Engineer prepared the evidence were State-wide valuation, rate study, and promulgation of operating rules for the Pacific Telephone and Telegraph Company; valuation and rate study of the Seattle Lighting Company; valuation of the street railway, light, and power systems of the Puget Sound Traction, Light, and Power Company; and valuation and rate study of the Puget Sound Navigation Company.

During the latter part of 1917 Mr. Phipps was commissioned a Captain in the Engineer Corps. He was called into training at Camp Lee, Peters-

* Memoir prepared by Frank F. Sinks, M. Am. Soc. C. E.

burg, Va., on January 1, 1918, at which time he resigned his position as Chief Engineer of the Public Service Commission of the State of Washington. Due to defective hearing he was not permitted to go overseas. In March, 1918, Captain Phipps was detailed to duty at Governors Island, New York, as Assistant to the Engineer Officer in charge of construction under the New York Depot Quartermaster. In April he was put in charge of all construction work on Governors Island as well as work of like character in New York City and vicinity. The construction work on Governors Island consisted of a large number of warehouses, including sprinkling systems, railroad tracks and yards, locomotive roundhouse, a ferry transfer bridge, a system of mains, stand-pipes, and pumping station for fire protection, fresh-water supply, and all other work necessary for a military depot. In addition to the work on Governors Island, Captain Phipps had charge of the repairs and alterations of many buildings in New York City and vicinity that were used by the Depot Quartermaster for various purposes. One who was closely associated with him at Governors Island says:

"Captain Phipps proved himself a very able engineer. His work was well systematized and his judgment with unusually good common sense were strong factors in the results obtained. He was forceful in his work and eminently fair to everybody, but was always careful to see that the Government received its due in labor and materials furnished. His work was a credit to himself and to the Engineer Corps."

Captain Phipps was mustered out of the Army on February 5, 1919, and entered private practice in Seattle, specializing in public utility valuation and rates. In 1923 the City of Seattle employed him as its Engineer in contesting the proposed rate increase of the Pacific Telephone and Telegraph Company. In 1924 the City of Seattle again employed him when the Seattle Lighting Company reduced its British thermal unit standard of gas. The Puget Sound Telephone Company, a pioneer in the use of time-measuring devices to base rates for telephone services, became involved in a controversy, and employed him as its expert Engineer. He also made studies of rates for a number of public utility corporations operating in the State of Washington and in British Columbia.

In the latter part of 1925, he was employed by the Tax Commission of the State of Washington, but due to failing health was unable to render much service and was obliged to resign.

On February 13, 1913, Mr. Phipps was married, in Seattle, to Mrs. Jennie C. Trimnell, who, with one child, Jane Evelyn, survives him.

Mr. Phipps was known among his associates as a man having the highest ideals of professional ethics, of sterling integrity, and a staunch friend.

He was a Mason, having been a member of the Blue Lodge, Scottish Rite, and Shrine. He was also a member of the Seattle College Club.

Mr. Phipps was elected an Associate Member of the American Society of Civil Engineers on October 3, 1911, and a Member on November 10, 1915.

WILLIAM GALT RAYMOND, M. Am. Soc. C. E.*

DIED JUNE 17, 1926.

William Galt Raymond was born on March 2, 1859, at Princeton, a little town on the Iowa side of the Mississippi River, about 20 miles above Davenport. Through his father, William Henry Vining Raymond, and his mother, Laura Guernsey (Peet) Raymond, he was descended from families prominent in Connecticut history from Colonial times. One of these ancestors, seven generations back, Capt. Thomas Yale, with his stepfather, Theophilus Eaton, came from England to New Haven in 1637. His father's work in education took the family much about Illinois and developed in the lad a variety of interests. His mother's frailty of constitution had an important influence in leaving him to his own devices. Far and wide he roamed over many a countryside, and early earned a reputation for vigor, self-reliance, and fertility of resource. One of his earliest interests was in watching machinery and in conducting mechanical experiments.

After assiduously attending the public schools in various cities of Illinois and Kansas, he went from his home in Leavenworth, Kans., to the University of that State, where for three years (from 1877 to 1880), in conformity with his father's wishes, he took a course in arts. Subsequently (1881-82 and 1883-84) he completed the Civil Engineering course at Washington University, St. Louis, Mo., from which he was graduated in 1884 with the degree of Civil Engineer. In 1905, he received the degree of Doctor of Laws from this institution. In 1919, he was honored by the University of Michigan with a Doctorate in Engineering.

Mr. Raymond's practical engineering experience began during his summer vacations and the years that intervened between his periods in college. He was successively engaged upon preliminary and construction surveys for the Kansas Pacific Railroad, now a part of the Union Pacific; and for the Kansas City, Fort Scott, and Gulf Railroad, and the Springfield and Memphis Extension of the latter. For a time he was a Resident Engineer in charge of construction.

After a brief period on Mississippi River improvement work, he was appointed, in 1884, an Instructor in Civil Engineering in the University of California. In 1890, he resigned to take charge of the location, under the late Samuel Rockwell, M. Am. Soc. C. E., Chief Engineer, of a railroad through the Coast Range Mountains between the San Joaquin Valley and San Francisco Bay. Subsequently, he engaged in general private civil engineering practice, including land, mining, and water-supply surveys, and worked on numerous surveys and plans for drainage, irrigation, and town sites. For a time he was Town Engineer of Berkeley, Calif., planning and executing a considerable amount of municipal work.

From 1892 to 1904, he served as Professor of Geodesy, Road Engineering, and Topographical Drawing at Rensselaer Polytechnic Institute, Troy, N. Y.

* Memoir prepared by S. M. Woodward, and R. B. Kittredge, Members, Am. Soc. C. E.

During this period he was Chief or Consulting Engineer for the Troy Department of Water Supply, designing and building the greater part of the present water supply system of that city. He also built the Troy-New England Electric Railway, and was Consulting Engineer in many cases involving questions of water power and mill rights.

In 1904, he went to the Iowa State University at Iowa City as Professor of Civil Engineering, and, in 1905, was appointed Dean of the newly created College of Applied Science at the same institution, which position he held until his death. In his later years Dean Raymond gave much time to the legal and economic problems of public utilities and wrote extensively in this field. For five years he was a member of the Special Committee on Valuation of Public Utilities of the Society, during which period his industry, energy, and judgment were of great assistance in preparing its report.

Besides nearly one hundred papers published in technical journals and society proceedings, he was the author of: "Plane Surveying" (1892); "Elements of Railroad Engineering" (1908); "Railroad Field Geometry" (1910); Railroad Field Manual for Civil Engineers (1915); section on "Surveying" of Mark's Mechanical Engineers' Handbook (1916); "What is Fair?" (1917); and "The Public and Its Utilities" (1925).

Dean Raymond was always active in matters of professional engineering organization. He was a member of the Society for the Promotion of Engineering Education from its organization in 1893, having served as President in 1911-12; an active member and Past-President (1913-14) of the Iowa Engineering Society; and a member and Past Chapter President (1915-16) of Sigma Xi. He was also a member of the American Railway Engineering Association; the Rensselaer Society of Engineers; the Chicago Engineers' Club; Phi Kappa Psi; Triangle; and Tau Beta Pi. He was affiliated with the Elks, and with the Masonic order, in which he held high rank, having been Past Master of Iowa City Lodge No. 4; a Shriner; a Knight Templar; a Thirty-second Degree Mason; and a Knight Commander of the Court of Honor.

In his community Dean Raymond was an active citizen and leader, a Director in various corporations, a Director of the Chamber of Commerce, a member of the Rotary Club, a Trustee and, for several years, Sunday School Superintendent of the Presbyterian Church, and, for six years, a member of the School Board. To the duties of the latter position he gave unstintingly of his time throughout an arduous period during which most of the old school buildings of the city were replaced by modern structures.

Ever an ardent lover of music, Dean Raymond was for eight years Manager of an Annual Concert Course in Iowa City, for which he secured the highest professional talent. When the University radio broadcasting station was opened, the responsibility of its operation was assigned to him, and to the development of its educational and recreational features he gave much personal attention.

At Fort Scott, Kans., on July 1, 1885, Dean Raymond was married to Helen Williams, daughter of Edwin R. Bay, a pioneer druggist, of Chicago,

Ill. Of this marriage four children were born: Margaret Leonard, wife of Harry P. Hammond, M. Am. Soc. C. E.; William Yale, Sales Superintendent; Edwin Bay, Banker; and Laurence Guernsey (deceased). For the last few months before his death Dean Raymond had not been in the best of health. In May, 1926, he entered the University Hospital for an operation, after which he appeared for about two weeks to be making excellent progress, when an embolus in the lung quickly brought on the end. Mrs. Raymond and three children survive him.

Perhaps Dean Raymond's deepest influence was exerted through his teaching and through the local affairs of the communities in which he lived. As an educator he was unusually progressive and much in advance of the general ideas of his time. In engineering college education he was an early and always an energetic advocate of broad training in fundamentals as contrasted with specialization. He believed enthusiastically in recognizing individual differences among students and, as far as he was able, adopted scholastic methods to permit of individual variations in the rate of doing college work. His intimate association with his students, his sympathy with them, and his devotion to their interests attracted them to him in a remarkable way. In his innumerable contacts with students over matters of discipline and the conduct of college activities, his sterling and outspoken courage and sincerity, combined with his high ideals of accomplishment, wielded an influence that permeated the entire student body. He touched the lives of students and penetrated their aims and ambitions to an extent impossible to measure.

Dean Raymond endeared himself to his associates in a way that passes on his influence to the future in an outstanding degree. Honesty and fairness, conscientious industry, and intellectual vigor were combined in him with an incessantly active nature, a hearty frankness, and a tender sympathy. All these inspired a deep loyalty and affection among his colleagues and students alike. With his departure an outstanding personality is lost, but the beneficent influence of that life and character will be felt and cherished for long years to come.

Dean Raymond was elected a Member of the American Society of Civil Engineers on October 3, 1894.

CHARLES BRADLEY ROWLAND, M. Am. Soc. C. E.*

DIED NOVEMBER 1, 1926.

Charles Bradley Rowland was the son of the late Thomas Fitch Rowland, Hon. M. Am. Soc. C. E., and Mary Eliza Bradley Rowland, and was born in the Greenpoint Section of Brooklyn, N. Y., on May 22, 1863. He received his early education in private schools in Brooklyn and New York, N. Y., and later entered the School of Mines, Columbia University, from which he was graduated in 1884, with the degree of Civil Engineer.

Immediately thereafter he entered The Continental Iron Works which had been carried on by his father for many years, and began his career in the machine shop. This was followed by several years' experience in the drafting room, and, later, he became Vice President of the Company, which position he occupied at the time of his death. As such, he exercised a general supervision over the Company's activities, having been particularly connected with the building of illuminating gas producing plants, supervising the construction of refrigerating machinery, and also special machinery used in the production of paper pulp.

Mr. Rowland was a member of the American Society of Mechanical Engineers, the Society of Naval Architects and Marine Engineers, and was also a member of the Union League Club of New York and the Franklin Institute of Philadelphia.

He was married in 1903 to Alice Cary of Detroit, Mich., who survives him with a son, Thomas Fitch Rowland, 2d, and a daughter, Mary Esther Rowland. He died at his home in Greenwich, Conn., on November 1, 1926, after a brief illness.

Mr. Rowland was elected a Member of the American Society of Civil Engineers on May 1, 1895.

* Memoir prepared by Thomas Fitch Rowland, Jr., M. Am. Soc. C. E.

RICHARD WILLETTE SHERMAN, M. Am. Soc. C. E.*

DIED MAY 23, 1926.

Richard Willette Sherman, the eldest son of Gen. Richard U. and Mary F. Sherman, was born at Utica, N. Y., on December 4, 1848. He attended the public schools of Utica in 1856 and 1857. In the spring of 1858 the family moved to a farm in the adjoining Town of New Hartford where he attended the district school and, later, during two winters, Bryant and Stratton's Commercial School, in Utica.

During 1864 and 1865, on account of the absence of his father, who was Assistant Clerk of the House of Representatives at Washington, D. C., he was in charge of the farm.

In 1866, Mr. Sherman secured a position as Rodman in the Engineer Corps of the Utica, Chenango, and Susquehanna Valley Railroad Company (now the Delaware, Lackawanna, and Western Railroad Company) between Utica and Sherburne, N. Y. In connection with this work and subsequent employment, and with the assistance of his superiors, he studied civil engineering, devoting as much time as possible to this task. In the spring of 1867, he accepted a similar position with the Albany and Susquehanna Railroad Company (now the Delaware and Hudson Railroad Company) between Bainbridge and Binghamton, N. Y., having been attracted by the tunnel work then in progress on that line.

From 1868 to 1872, he held the position of Assistant Engineer for the New York and Oswego Midland Railroad Company (now the Ontario and Western Railroad Company), in charge of surveys and the construction of the road between Oneida and Central Square, N. Y. On the completion of this work, he was appointed Resident Engineer of the Company's line between Walton and East Branch, N. Y., including the Hawk's Mountain Tunnel. During 1870, he was also Assistant Engineer in charge of construction on the New Jersey Midland Railroad.

In 1873, he accepted the position of Assistant Engineer in charge of construction and surveys of all new lines for the Erie Railroad, but resigned in 1875 to go into the contracting business on the New York and Canada Railroad, near Pittsburgh, Pa. In 1876, Mr. Sherman, in partnership with Dr. Emmett Flagler, constructed the West Troy and Green Island Water-Works, as well as water-works plants at Gloversville, Richfield Springs, and Walton, N. Y. The firm also constructed public works in Vermont and Ohio, and made improvements on the Champlain Canal for New York State.

The firm of Sherman and Flagler was dissolved in the fall of 1879 when Mr. Sherman went to Cuba to build a gas plant to supply the entire city of Havana. He returned to the United States in 1880 to construct by contract the water-works for Greenwich, Conn.

* Memoir compiled from information on file at the Headquarters of the Society.

In 1881, Mr. Sherman, in partnership with Mr. Michael McDonough, secured a large contract to construct a number of miles of the West Shore Railroad, near Haverstraw, N. Y. This work was completed in 1882 and the firm of Sherman and McDonough continued an extensive and successful contracting business, constructing water-works at Lansingburgh, Sandy Hill, Little Falls, Camden, Canastota, New Berlin, and Oneonta, N. Y., as well as sewers at Lansingburgh and Amsterdam, N. Y., and South Framingham, Mass., together with canal work under numerous contracts for the State of New York.

From 1886 to 1888, Mr. Sherman served as City Surveyor of Utica. While in this position, and with the co-operation of Mayor T. E. Kinney, the public works of the city were rapidly brought up to date. The program laid down by Messrs. Sherman and Kinney has been continued, the City of Utica spending on an average about $250 000 per year on pavements, sewers, etc.

In 1888, the firm of Sherman and McDonough was incorporated as the Troy Public Works Company with a capital of $30 000, and continued to carry on a successful contracting business with Mr. Sherman as its President. In 1896, Mr. McDonough retired when the Company moved its home office to Utica. Mr. Sherman's brothers, the late Hon. James Schoolcraft Sherman, a member of Congress and, later, Vice-President of the United States under the Taft Administration, and Mr. Sanford F. Sherman, succeeded Mr. McDonough as partners in the business.

The Troy Public Works Company constructed a large part of the water-works for Utica and Norwich, N. Y., as well as public works for Perry and Amsterdam, N. Y., New Britain, Conn., Boston, Pittsfield, and North Adams, Mass., and considerable State work along the Eastern Division of the Barge Canal. The Company also built under contract with the Government the foundation for the United States Gun Factory at Watervliet, N. Y.

Mr. Sherman served as Democratic Mayor of Utica during 1900 and 1901 and again in 1906 and 1907. Under his first régime, $165 000 was appropriated for a new public library and large sums for new school buildings. Provision was made also for new river bridges, for the abolition of railroad grade crossings on the principal streets, for a new union passenger station, a new New York Central Railroad freight house, etc.

On retiring from his second term of office as Mayor of Utica, Mr. Sherman resumed his work with the Troy Public Works Company and completed the plant of the Empire State Power Company, on Schoharie Creek, near Amsterdam, N. Y. From August, 1911, to April, 1915, he held the important position of Chief Engineer of the New York State Conservation Commission.

Mr. Sherman was self-reliant and held strong opinions generally based on firm foundations. Thoroughness was one of his outstanding characteristics and in early manhood he acquired the best and most practical education obtainable. In his railroad building he gained an experience and knowledge of methods that combined to make him a skilled engineer. As a constructor of public works, his thoroughness and his faithful adherence to contracts brought him deserved success. At all times he stood for the best in civic, social, and

educational life. As an Engineer, and as Mayor of Utica, he gave to his work his best judgment, his most careful consideration, and his honest endeavor.

He was a member of Utica Lodge 47, F. and A. M., the Oneida Historical Society, and the Albany Country Club. He was also a member of the First Presbyterian Church of New Hartford.

On May 20, 1873, he was married to Julia H. Hammond, who died on March 13, 1919. He is survived by a daughter, Elizabeth Sherman, a brother, Sanford F. Sherman, and a sister, Mrs. Henry J. Cookinham.

Mr. Sherman was elected a Member of the American Society of Civil Engineers on October 6, 1886.

WALTER AUGUSTUS SUMNER, M. Am. Soc. C. E.*

DIED JUNE 25, 1926.

Walter Augustus Sumner was born in Unionville, Mo., on December 17, 1884, the son of Horace Augustus and Ida May (Holbrook) Sumner. He was the youngest of three sons, and was educated in the grammar school and in the East Denver High School after his parents had taken up their residence in Denver, Colo. in 1886. He entered Leland Stanford Jr. University, in California, in 1904, remaining in that institution until the earthquake in 1906. He was made a member of the Sigma Alpha Epsilon Fraternity while at Stanford.

Following the severance of his connection with Stanford University, and until the latter part of 1908, Mr. Sumner was employed on several railroad surveys, as Chainman, Rodman, and Levelman, principally as Levelman on the Denver, Northwestern, and Pacific Railway (now the Denver and Salt Lake Railway), under the direction of his father, the later H. A. Sumner, M. Am. Soc. C. E., who was Chief Engineer of the Company.

During 1909, he served as Locating Engineer on the Green River Canal Project in Wyoming, defining the alignment and grades of a canal 72 miles in length designed to carry 300 sec-ft. of water. In 1909, Mr. Sumner was also Assistant Engineer on the location of a revised alignment of the Denver and Rio Grande Railroad from Soldier Summit, Utah, westerly, where a 2% grade was adopted to take the place of the 4% grade then in operation. This work was under the direction of James G. Gwyn, Chief Engineer.

From the latter part of 1909 until the summer of 1915 he was employed by the late D. G. Thomas, M. Am. Soc. C. E., Chief Engineer and General Superintendent of the Denver Union Water Company, as Engineer and Superintendent in charge of properties of that Corporation outside the City of Denver. His duties involved the construction, repair, and maintenance of a water plant and system supplying Denver with a maximum of 87 000 000 gal. per day of filtered and treated water. A thorough knowledge of the construction and operation of slow sand, mechanical, and rapid sand filters was acquired by him in this position, as well as a knowledge of the sterilization of filtered water.

During the period of Mr. Sumner's connection with this water system the construction of one diversion dam and sand trap in the bed of the South Platte River, and the installation of eleven miles of cast-iron, steel, and wood-stave pipe conduits up to 60 in. in diameter, were a part of the construction work under his charge.

In 1915 and 1916, he was engaged in research work connected with water sterilization, and in the development of a product known as "Sterilall".

During a part of 1916 he was employed by G. W. Harris, M. Am. Soc. C. E., Chief Engineer of the Coast Lines of the Atchison, Topeka and Santa Fé Railway Company, near Los Angeles, Calif., in charge of the construc-

* Memoir prepared by the late Horace A. Sumner, M. Am. Soc. C. E.

tion of plain and reinforced concrete structures, and on matters connected with valuation of the railway. During the latter part of that year and the first half of 1917 Mr. Sumner was also employed by Mr. Harris to superintend the construction of a reinforced concrete multiple-arch dam, 150 ft. high, near Escondido, Calif., with 10 miles of 42-in. concrete distributing conduits.

Mr. Sumner took part in the World War, being commissioned by the War Department as First Lieutenant of the Engineer Reserve Corps on July 31, 1917. On December 28, 1917, he was ordered into service and received engineering training at Camp Lee, Petersburg, Va., where he remained about two months. He was then ordered to report to the officer in charge of the Cantonment Division, Quartermaster Corps, Washington, D. C., which later became the Construction Division of the Army.

After remaining in Washington until May 18, 1918, Lieutenant Sumner was detailed to supervise water and sewerage plants and the installation of plumbing and steam heating at Camp Perry Proving Grounds, Fort Clinton, Ohio. He also performed the duties of "Disbursing and Property Officer". While stationed at Camp Perry he received his promotion as Captain, Quartermaster Corps, U. S. Army, his commission bearing the date of August 24, 1918.

On September 14, 1918, he was ordered to report to Willoughby, Ohio, where he took station as Constructing Officer in charge of the construction of barracks and mess halls, with a hospital unit, to accommodate the housing of 1 000 men engaged in the Chemical Warfare Service.

On the completion of this work, on November 21, 1918, he again reported to Camp Perry Proving Grounds, where he resumed his duties as Assistant to the Constructing Quartermaster. He was relieved of these duties on August 12, 1919, being then ordered to Peoria, Ill., where he was Constructing Quartermaster in charge of work at the Holt Manufacturing Company. He remained in Peoria until November 18, 1919, when he was ordered to duty as Constructing Quartermaster of the Picatinny Arsenal, near Dover, N. J., his work consisting of the construction of eleven additional buildings for the manufacture and storage of high explosives.

On September 25, 1920, Captain Sumner received his honorable discharge from the Army. It was a source of keen disappointment to him that he was not permitted to serve his country on foreign soil, but the wisdom of his retention on this side was proved by the valuable service which his engineering training rendered him capable of performing in lines of work so briefly described herein. His true patriotism was clearly shown by the cheerful spirit in which he bent his energies to the supply of the sinews of war to the comparatively few engaged in actual combat.

During a part of 1920 and until the middle of 1922, Mr. Sumner was engaged in private practice in New York, N. Y. Most of this time was given to the design of structures and to a general layout plan for the Fulton Lumber Company Terminal near Hoboken, N. J.

In 1922, and until April, 1924, he was engaged in consulting engineering and general contracting activities in Los Angeles and San Francisco, Calif.

From April, 1924, to August, 1925, he was employed by the Denver Municipal Water-Works, under the direction of R. S. Sumner, M. Am. Soc. C. E., General Manager, as Constructing Engineer in charge of the construction of a new and modern rapid sand filter plant at Marston Lake, near Denver. The capacity of this plant assures the filtration of a maximum of 64 000 000 gal. of water per day, and its cost was in the neighborhood of $2 000 000. After the completion of that work he was engaged to the time of his death in general practice as Consulting Engineer and Contractor in Denver.

Mr. Sumner was adapted by nature for the profession of an engineer. He was gifted with a mind which prompted a close scrutiny into every detail of the work in hand, and made him conversant with the reasons for or against any mode of procedure. He was especially gifted with the faculty of handling men and attending to the assembling of materials to be used, thus avoiding unnecessary delays and extra costs which follows so often in works of magnitude.

Of a genial and happy disposition, he was a stimulus to all employees under him, and was able to control and carry on the work in hand with the loyal support of all members of his staff. Had he lived longer, the inherent gifts with which he was endowed and the further experience in professional activities would undoubtedly have placed him among the brilliant and useful members of the Engineering Profession.

Socially he had an extensive acquaintance and his inborn wit and good nature were the delight of any gathering. He was married on October 27, 1909, to Jessie May Tucker, daughter of Mr. and Mrs. Cromwell Tucker, of Denver, Colo. His home life was happy, one which could be envied by all.

Mr. Sumner was elected a Member of the American Society of Civil Engineers on January 17, 1921.

FRANK SUTTON, M. Am. Soc. C. E.*

DIED JULY 22, 1926.

Frank Sutton, the son of Peter and Jane (Mechling) Sutton, was born in Indiana, Pa., on May 14, 1859. At the age of 20, he entered the Pennsylvania Military College at Chester, Pa., from which he was graduated in 1879 with the degree of Bachelor of Science in Civil Engineering. In 1912, the degree of Master in Civil Engineering was conferred on him by the same institution, and, in 1916, he was elected a Trustee of the College.

Following his graduation, from 1879 to 1886, Mr. Sutton was employed by the Pittsburgh and Western Railroad Company, the old "South Penn" as it was then known, serving as Draftsman, Rodman, Transitman, and Topographer. The "South Penn" Corps set a standard in topographic work that has seldom been equalled, and Mr. Sutton gained topographic experience that afterward brought to him the high position he held in that branch of engineering in the United States Geological Survey.

On April 5, 1886, Mr. Sutton was appointed a Draftsman in the U. S. Geological Survey and, in 1889, he was placed in charge of a topographic field party. His natural ability as a Topographer, combined with his training as an Engineer, attained for him rapid promotions and, in 1906, he reached the grade of Geographer in charge of the Atlantic Division of Topography. It was in this section of the United States, that reaches from Maine to Florida and includes Ohio with the States to the south and west as far as the Mississippi River, that he developed the men under his direction and produced topographic maps of the highest order which are a lasting memorial of his ability.

Frank Sutton was commissioned a Major in the Engineer Officers' Reserve Corps on January 23, 1917, and was called to active duty on June 22, 1917. He was first assigned to special topographic mapping duty with the U. S. Geological Survey, but later was assigned to duty with the 25th, 29th, and 604th Engineers, having had command of the latter. He served in France from August, 1918, to April, 1919, and on the completion of his service with the American Expeditionary Forces, he returned to the United States and was honorably discharged on April 7, 1919. He was then reinstated in his former position of Geographer in the U. S. Geological Survey, which position he held until his retirement, on account of ill health, on March 7, 1924.

Major Sutton's death was probably the result of exposure and sickness at Camp Devens, Massachusetts, during the winter of 1917-18. After a long period of treatment at Johns Hopkins University, where his ailment was diagnosed as amyotrophic lateral sclerosis, he was taken to his home at Indiana, Pa., where for more than two years he received the loving care of his sister and two brothers. His condition gradually grew worse until at the time of his death he was completely paralyzed, although he retained to the end, his mental faculties, his cheerful disposition, and his keen interest in current events.

* Memoir prepared by C. H. Birdseye, M. Am. Soc. C. E.

Major Sutton's stability of character, devotion to duty, shrewd wisdom, and fine sense of humor made for him a host of friends by whom his loyalty and kindly advice will be remembered with deep affection. His death brought to an end a life of constant devotion to his chosen profession of engineering.

Major Sutton never married. A sister, Mrs. W. F. Wettling, and two brothers, Ross M. and Phil M. Sutton, of Indiana, Pa., survive him.

His membership in societies and clubs included the Cosmos Club and Chevy Chase Club, of Washington, D. C., Society of American Military Engineers, Washington Society of Engineers, Geological Society of Washington, American Association for the Advancement of Science, American Forestry Association, and American Federation of Arts.

Major Sutton was elected a Member of the American Society of Civil Engineers on October 3, 1906.

NORTON LONGSTRETH TAYLOR, M. Am. Soc. C. E.*

DIED FEBRUARY 21, 1926.

Norton Longstreth Taylor the son of William Curtis Taylor, was born in Philadelphia, Pa., on February 10, 1861. His earliest known ancestor in America was Thomas Scattergood, who came to this country early in the Seventeenth Century from England. His mother was Elizabeth Longstreth, of the fifth generation of the family in this country, the first of whom was Bartholomew Longstreth who emigrated from Yorkshire, England, in 1699.

Mr. Taylor received his engineering training by being in actual contact therewith from his early manhood and by close application to the study of mathematics, and particularly to the science of civil engineering.

In his youth he attended the Friends' School and, later, the University of Pennsylvania for one year, after which he served an apprenticeship with the Baldwin Locomotive Works from June, 1879, to February, 1881. He then entered railroad work with the Texas-Mexican Railway Company, as Chainman, Topographer, and Levelman until March, 1882.

Between March and September, 1882, he finished his machinist apprenticeship with the Baldwin Locomotive Works and in October, he entered the service of the Pennsylvania Railroad Company as a Draftsman. He continued with this Company on various office and field works, until September, 1890, and during the last four years of this period he served as Assistant of Maintenance of Way.

While an employee of the Pennsylvania Railroad Company, Mr. Taylor was Inspector on three large stone masonry arches, and Assistant Engineer on the construction on three miles of track of the Morris River Branch and eleven miles of track, with pile trestle and large wharf, on the West Jersey Railroad Branch. He ran 800 miles of precise levels over the lines of the Pennsylvania Railroad Company achieving remarkable accuracy in his instrument work. He was also identified with the construction of the 100 000-ton coal stocking plant at South Amboy, N. J., the re-building of Pier No. 4 in the same place, the construction of Becks Run and Streets Run Viaducts on the Monongahela Division, the revision of one mile of line on the Maryland Division, and other work of the Company.

Mr. Taylor gave up railroad work in 1890 to better himself financially, joining the Tacoma Light and Water Company as Assistant Engineer on surveys, plans, and estimates for a proposed gravity water system for the City of Tacoma, Wash. He was retained on this work until April, 1891. From this date until January, 1893, he was engaged in private practice on mining and water-power projects, surveys, etc. In January, 1893, he began work as Computer and Draftsman on the geographical location of harbor lines and tide-land areas for the State of Washington, on which he continued until February, 1895.

* Memoir prepared by W. A. Kunigk, M. Am. Soc. C. E.

From March, 1895, to September, 1897, he was employed by the Northern Pacific Railway Company as Transitman, Draftsman, and, also, in charge of field work on the revision of alignment and grade in Montana. During the period from September, 1897, to April, 1898, he held the position of Chief Assistant Engineer of the Chilkoot Railway and Transport Company on the survey and construction of an aerial tramway over Chilkoot Pass.

In April, 1898, Mr. Taylor was appointed City Engineer of Tacoma. During his tenure of office he built the first continuous wood-stave pipe in Tacoma, designed and built a 4 500 000-gal. reservoir, made about 45 miles of trunk and lateral sewer extensions, and improved many districts by grading, paving, and sidewalks. He retired from office in January, 1904, but was again retained as Assistant City Engineer from August, 1904, to March, 1907, during which time he designed many improvements in the water and sewerage systems. From April, 1907, to July, 1908, he served as Chief Engineer for the Metropolitan Park District of Tacoma, laying out a boulevard system, approximately 20 miles in length, 5 miles of which were constructed during his term in office.

From August, 1908, until January, 1926, Mr. Taylor was continuously in the employ of the Light Department of the City of Tacoma. Until June, 1916, he was engaged as Engineer on surveys for extensions, maps, office records, and Department statistics. Subsequently, he worked for about three years as Senior Engineer, collecting data and preparing maps and estimates in reference to different power sites available to the City of Tacoma. He then carried on the preliminary investigations in reference to the Lake Cushman power site, and his untiring work in this connection and devotion to his duty gradually broke down his health.

Mr. Taylor's advice was very largely instrumental in convincing the city officials of Tacoma of the economic and practical potential possibilities of the Lake Cushman power development in preference to others that were considered at that time. During the actual development work of the project, Mr. Taylor held the position of Superintendent of Surveys, until January, 1926. He then went on leave of absence to Glendale, Calif., to recuperate his rapidly failing health, where, after an operation from which he never recovered, he died on February 21, 1926.

Mr. Taylor was a gentleman of the quiet, modest type, whom one could only learn to appreciate by years of contact. Those who loved him most were the ones who had known him longest. His utter unselfishness, sincerity of purpose, and the philosophical viewpoint that he applied to every problem, made his advice and counsel always welcome to his associates. In his work, he was thorough and untiring, with an infinite patience in attention to details, always giving unsparingly of himself to his professional duties which, no doubt, was the chief cause of undermining his rugged vitality and hastening his untimely death.

Mr. Taylor was a great lover of good music and a devout student of philosophy and Masonry. These characteristics, together with his charming personality, made his loss irreparable to those who were closely associated

with him. He was a Thirty-second Degree Mason, a member of Lebanon Lodge No. 104, and was elected to the Scottish Rite Bodies on November 15, 1910.

He was married to Ora B. Jones, of Evansville, Ind. Two daughters, Allison and Phyllis, were born to this marriage. He is survived by his widow and his elder daughter, Allison.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on February 4, 1891, but resigned his membership on December 31, 1904. He was again elected a Member on May 28, 1923.

EDWIN HOWARD THOMES, M. Am. Soc. C. E.*

DIED OCTOBER 29, 1926.

Edwin Howard Thomes was born on April 5, 1875, in Rowayton, Conn., the son of Charles F. and Mary (Godfrey) Thomes. He was a descendant of Charles Chauncey, second President of Harvard University from 1654 to 1671.

Following his education in the public schools, Mr. Thomes attended the High School at South Norwalk, Conn. In 1894 he received the degree of Bachelor of Philosophy at Yale University, and in 1895 that of Civil Engineer from Columbia University. He was a member of the Phi Delta Theta Fraternity.

Mr. Thomes served as Assistant Engineer with the New York State Engineer and Surveyor's Department on canal construction from February, 1896, to September, 1898, in charge of 7 miles of Erie Canal improvement, the cost of which was about $400 000. This work included the construction of a new steel highway bridge, a concrete dam, concrete steel culverts, underpinning and raising bridges, walls, etc.

From September, 1898, to April, 1899, he was engaged in private practice at Syracuse, N. Y., and made the surveys for and designed the sewerage system of Weedsport, N. Y. He was associated with Belden and Seely, Contractors, on the construction of an electric railway from Albany to Hudson, N. Y., in April and May, 1899.

From June to September, 1899, Mr. Thomes was in charge of the surveys for and the location of 30 miles of an electric railway for the Houghton County (Michigan) Railroad Company; and from September, 1899, to May. 1900, he was with the United States Corps of Engineers, under Col. J. W. Barlow, U. S. A., in charge of various surveys, deep borings, etc., including plans and estimates for extending the navigation of the Passaic River to Paterson, N. J., by canal, locks, dams, etc. Mr. Thomes was also in charge of dredging and rock removal in the Passaic River and Raritan Bay, New Jersey.

From May, 1900, to October, 1904, he was with the Rapid Transit Commission of New York City, Section 1, as Assistant Engineer in charge of field work and part of the time in charge of the Section. From October, 1904, to August, 1905, he served in the Topographical Bureau, Borough of Queens, New York, N. Y., in charge of various surveys, monumenting, triangulating, etc., and from August, 1905, to his death on October 29, 1926, in the Bureau of Highways in charge of surveys, plans, specifications, estimates, investigations, reports, etc., and the construction of pavements, sidewalks, highway bridges, and culverts. To all these duties Mr. Thomes devoted his utmost energies; no work was too difficult or too laborious. He had a faculty for solving the most difficult problems which many others could not seem to fathom.

* Memoir prepared by Herman K. Endemann, M. Am. Soc. C. E.

He was well liked by the men with whom he was associated and was considered an authority on all matters pertaining to highway construction. He was appointed by the late Mayor Gaynor, of New York, to represent the city as a delegate to the International Road Congress held in London, England, from June 23 to June 28, 1913.

Mr. Thomes was a member of the Municipal Engineers of the City of New York and the American Road Builders Association. He was also a member and Treasurer of the Jamaica Club, Jamaica, N. Y., in which he took a most active interest, as well as a veteran of the Volunteer Fireman's Association, of Jamaica.

He was a member of Jamaica Lodge No. 546, F. and A. M., and of Central City Commandery, Knights Templars, of Syracuse, N. Y.

He is survived by his widow, Mrs. Cora (Lake) Thomes, to whom he was married in 1900, by a daughter, Mrs. Wilma (Thomes) Holt, and by a granddaughter, Joyce Holt.

Mr. Thomes was elected an Associate Member of the American Society of Civil Engineers on December 6, 1899, and a Member on December 5, 1911.

HENRY CLARK THOMPSON, M. Am. Soc. C. E.*

DIED APRIL 13, 1926.

Henry Clark Thompson was born in Troy, N. Y., on September 26, 1861, the third son of John Calhoun Thompson, a Captain in the Civil War, who was the grandson of Daniel Thompson, a soldier of the American Revolution, and a lineal descendant of earlier settlers in America. His mother, whose maiden name was Anna Thompson, was also a member of an early American family, not however related in any way to the Daniel Thompsons.

In the early part of 1862, the City of Troy was partly destroyed by fire, and the Thompson family moved to New York, N. Y. At the age of eight, Henry Clark Thompson went to live on his father's country estate near Morristown, N. J., but returned to be graduated from a New York public school. From his father who was a builder, actively interested in the construction, at different times, of many houses in New York, Mr. Thompson inherited a taste for the structural work in which he, during the earlier years of his professional life, co-operated to some extent. Entering the employ of Maclay and Davies, Civil Engineers and Surveyors, of New York, the boy made good to an exceptional degree and, in his nineteenth year, was placed in charge of the building of the large iron pier at Rockaway.

At twenty years of age the Western country lured him as it did many other young men of the period, and he accepted an offer to serve as Assistant Engineer on the construction of a part of the new Denver, Western, and Pacific Railroad. When this work was well advanced, Mr. Thompson turned to another attraction of the same kind and contributed his share to the building of the road now famous as the Atchison, Topeka and Santa Fé. About that time an urgent appeal from his father brought him back East, the appeal taking the tangible and practical form of an offer to put the young engineer through college. He was graduated from the Columbia School of Mines in 1886. The Faculty statements, still preserved, of his progress as a student reveal that the high grades of "nine" and "ten" were not uncommon throughout his four years of attendance.

Immediately after graduation, Mr. Thompson secured a position as Assistant Engineer with the Suburban Elevated Railroad of New York. This, like his previous work, was construction. He spent fifteen months in this capacity, and then accepted a position as Resident Engineer of the Orange Mountain Land Company of New Jersey, on a $500 000 plan, involving the layout and construction of streets, sewers, and water supply.

In 1889, Mr. Thompson had reached the point where he felt justified in going into business for himself. In the decade which followed, during a few years of which very little capital was risked in enterprise, he yet found more than enough to do. His work was varied, including highways, bridges, and pavements; water supply, sewers, and drainage; electric railway; and the erec-

* Memoir prepared by C. E. Chase, Asst. Div. Engr., M. of W. Dept., River Div., N. Y. C. R. R., Weehawken, N. J.

tion of sixty or more buildings. Also during this period, as City Surveyor and as Engineer of the Village of Williamsburg, N. Y., he conducted surveys of farms and estates, laid out and monumented 50 miles of city streets, and surveyed more than 10 000 city lots.

In 1900, Mr. Thompson became Supervisor of Bridges and Buildings for the New York Central Railroad Company (West Shore) at Weehawken, N. J., and shortly established a record for bridge building by completing more than 100 spans "under traffic" in 100 consecutive days. In 1902, he was appointed Division Engineer, his territory extending from Jersey City, N. J., to Albany, N. Y., and remained in resident charge of maintenance for twenty-four years, until the time of his illness and death.

He was a railroad and civil engineer of calm judgment and marked resource, as well as a man whose counsel was valued and sought. For himself, he desired no wider field of effort than to remain of service to his Company and its leading officers, and to set a good example for his subordinates to follow. Many of his co-workers, especially those who were nearest to him in the New York Central service, have felt very keenly the sense of personal loss.

About twenty years ago, Mr. Thompson was married to Mary Lawson Archer who, with their two children, a son and a daughter, survives him.

He was a Royal Arch Mason and a member of Eureka Lodge, New York. He also belonged to several railroad, engineering, and Columbia University clubs and societies.

Mr. Thompson was elected a Member of the American Society of Civil Engineers on February 4, 1903.

JOHN THOMSON, M. Am. Soc. C. E.*

DIED MAY 31, 1926.

John Thomson was born on October 25, 1853, in Fochabers, Scotland, the eldest son of Alexander and Elizabeth Hay Thomson, and was brought to America when a child.

He was educated in the common schools and under special instructors, with mathematics receiving marked attention. At first, a watchmaker, Mr. Thomson later became an engineer and inventor.

For about twenty-five years he was associated with the Colt Company, of Hartford, Conn., for which he invented and designed the printing, embossing and cutting, and creasing presses known to the trade as Colt's Armory Presses. Later, he designed other printing presses for his own Company, the John Thomson Press Company.

In the field of water meters he also became distinguished as Inventor and Manufacturer. The first of such inventions was the Disc Water Meter, made by the Thomson Meter Company, which he established. He later designed the Trident, Disc and Crest Water Meters, and organized for their manufacture the Neptune Meter Company, of Long Island City, N. Y., which is believed to be the largest maker of water meters in the United States.

Almost the last work on which Mr. Thomson was engaged was the design of another water meter known as the Zenith. He also invented a number of features in connection with electric furnaces which are now being applied in industry.

In 1877 he was married to Alice Elizabeth McKee, of Canandaigua, N. Y. She and their two sons and one daughter survive him.

He died in Brooklyn, N. Y., on May 31, 1926. A fine portrait of him is in the Engineers' Club, of New York, with those of other Past-Presidents.

Mr. Thomson was a member of the American Society of Mechanical Engineers, the American Institute of Mining and Metallurgical Engineers, the American Electro-Chemical Society, and the Franklin Institute. He was one of the early members, and for three years President, of the Engineers' Club of New York, and was a member of the Union League Club of New York; the Pilgrims Society; and the Royal Thames Yacht Club and the American Luncheon Club, of London, England.

Mr. Thomson endeared himself to his fellow members of the Society by those qualities which have come to be associated with the highest type of Scotch character—ruggedness of purpose, stern integrity, and active hatred of cant and insincerity in any form. This heritage was a pronounced factor in his life, and his love for Scottish things found expression in his activities as a member of the St. Andrew's Society and the Burns Society, the latter of which he served as President.

Mr. Thomson was elected a Member of the American Society of Civil Engineers on March 2, 1887. He served as Director of the Society from 1892 to 1894, and as Treasurer from 1895 to 1899.

* Memoir prepared by Edward E. Bartlett, Esq., New York, N. Y.

FREDERICK HOWARD TILLINGHAST, M. Am. Soc. C. E.*

DIED JULY 15, 1926.

Frederick Howard Tillinghast was born in Providence, R. I., on September 19, 1877. At the age of twenty-one he was graduated from Brown University with the degree of Civil Engineer. The year following his graduation (October, 1899-June, 1900), he took a Post-Graduate Course in Hydraulics and Sanitary Engineering at the Massachusetts Institute of Technology.

Mr. Tillinghast's first experience in engineering was in railroading. He worked on the location, construction, and maintenance of more than one line in the Eastern States. Subsequently, he became connected with the United States Geological Survey as Hydrographer, making investigations of certain streams in New York State with the idea of an enlarged water supply for the City of New York.

In 1904, he joined the United States Reclamation Service and was with that service for a number of years. During this period he worked on the Belle Forche Irrigation Project in South Dakota, on the location, design, and construction of canals. He also revised the cost accounting system of the Project. After completing a few short assignments Mr. Tillinghast was placed in charge of the construction of the East Park Dam on the Orland Project, California. His most important work with the Reclamation Service, however, was on the Lahontan Dam, Nevada. He began with the preliminary field investigations, worked on designs, and finally became Resident Engineer on construction. When this work was finished he severed connections with the Service and engaged for a time in private practice. War conditions, however, made business uncertain and, in 1918, he took a position with the Sutter Basin Company in California, directing surveys. Later, he was promoted to be Resident Engineer, and, in 1921, was appointed Chief Engineer, which position he held until his death.

The circumstances connected with Mr. Tillinghast's death were more than ordinarily pathetic. About three years ago Mrs. Tillinghast began to fail in health and, in spite of the best medical advice, gradually became a hopeless invalid unable to help herself in the smallest degree. It seemed at this time as if few men were more needed than Mr. Tillinghast. His work occupied him during the day, and when that was done he went home to play the part of nurse. His own health during the last months preceding his death had not been the best, but there was apparently no occasion for alarm. However, the end came for him without a moment's warning. He is survived by his invalid wife and two sons, the elder of whom is married and lives in Woodland, Calif., while the younger has been attending the Junior College in Sacramento, Calif.

It always seems a futile thing to eulogize a man after he has gone, but, because Mr. Tillinghast was an Engineer and had certain gifts or acquirements which are desirable in a member of that profession, it may not be out

* Memoir prepared by S. H. Searancke, Assoc. M. Am. Soc. C. E.

of place to mention these, since they redound to his professional credit. He had the ability of mixing easily with all classes of people and was on terms of familiarity with the men he met in his work, whether superiors, equals, or subordinates. This gift of friendliness gained for him the regard and confidence of his fellow workers. He had in no small degree that plainness or commonness of manner which forms such a strong link of fellowship between a man in charge of work and those who work for him, and which almost always commands the highest type of loyalty. When he first went to the Sutter Basin in a more or less subordinate position, he was familiarly known as "Tilly", and it is significant of the man's own attitude that, after his appointment as Chief Engineer, no one thought it necessary to call him by any other name.

He had also the faculty of organizing men and directing work. He had confidence in those who worked for him, and knew how to shift part of the responsibility from his own shoulders to the shoulders of his subordinates, allowing them to find their own solutions to their problems.

For his co-workers, fellow-members in the Society, and a host of others who knew and liked him, there is much sadness in the thought that a friend has departed and will never be heard from again. There is nevertheless a certain satisfaction in the opportunity to sketch thus briefly the life of a plain engineer, not the type of man who has made for himself a nation-wide reputation, or who has had big work named for him, but the type that is never advertised and rarely even mentioned outside the small sphere of his own activities—one who has gone about his work in a modest and conscientious manner, giving much thought to his profession, and but little thought to the gratitude or rewards that might be due him.

Mr. Tillinghast was elected an Associate Member of the American Society of Civil Engineers on May 1, 1907, and a Member on April 1, 1914. It is interesting to note that among those referred to in his application are the names of A. P. Davis, and John R. Freeman, Past-Presidents, Am. Soc. C. E., and F. H. Newell, M. Am. Soc. C. E. Mr. Tillinghast was a member of both the Sacramento and San Francisco Local Sections.

MARK ALBIGENCE WALDO, M. Am. Soc. C. E.*

DIED JULY 16, 1926.

Mark Albigence Waldo, the son of William Williams and Dorothy (Wood) Waldo, was born in Manitowoc, Wis., on August 1, 1859. His early education was obtained in the local public schools, and he was a member of the second class to be graduated from the Manitowoc High School. In 1876, he attended the Academy at Prattsburg, N. Y., and, in 1878, entered the University of Wisconsin and was enrolled in the Engineering Course.

Having completed his Freshman year at the University, he accepted work under the late Grenville Mellen Dodge, Hon. M. Am. Soc. C. E., whose headquarters at that time were at Helena, Mont., on the construction of the Union Pacific Railroad. It was while engaged on this work that Mr. Waldo had his initial experience in bridge building, some of the structures over the Gallatin River and the head-waters of the Missouri River on which he was engaged being particularly difficult from an engineering standpoint. This work resulted in his rapid development as an engineer, with the promise of high rank in the profession which his record amply verified during his years of active practice.

After a year of practical experience, he returned to the University of Wisconsin to complete his engineering course, and was graduated therefrom in 1884 with the degree of Bachelor in Mechanical Engineering.

In September, 1884, Mr. Waldo commenced the practice of engineering by accepting the position of Instrumentman and Pile-Driver Inspector on the construction of the St. Louis River Bridge, at Duluth, Minn., for the Northern Pacific Railroad Company. In 1892, he served as Assistant Engineer on the construction of the Memphis Bridge, at Memphis, Tenn.; and, in 1898, he was Resident Engineer for the Missouri Bridge and Iron Works. In 1899, he was placed in charge of construction by this Company. Mr. Waldo specialized in bridge building, particularly in foundations and substructures which gave an outlet for his great ability and resourcefulness, and during his active practice he was associated with engineers who were representative of the best men in the profession, among whom may be mentioned the late George S. Morison and Alfred Noble, Past-Presidents, Am. Soc. C. E.

In 1904, Mr. Waldo retired from the practice of engineering and entered the field of phosphate mining in Polk County, Florida, as General Manager of the Dominion Phosphate Company, with headquarters at Bartow, the center of the pebble phosphate fields of the State. At this time he declined an offer from the Florida East Coast Railroad Company to serve as Chief Engineer on the construction of the "Overseas Railroad" from Miami to Key West.

From the time of his becoming a "Floridian" by adoption and from choice, he made his home in Bartow, taking an active interest in civic affairs, with particular attention to the interests of the Public Library. He was a member of the Library Board from the time of its organization until his death, his work and his advice having been sincerely appreciated by all who came within

* Memoir prepared by W. Lacy Boyd, M. Am. Soc. C. E.

its influence. It was during his service as a member of the City Council of Bartow that the question of sidewalks came before that body for discussion. Mr. Waldo firmly insisted that a policy should be outlined and a system adopted, based on a comprehensive survey of drainage conditions, grades, and street center lines. This recommendation prevailed, but before volume construction could be started, it was found that the interest created was such that a complete system of street improvement was demanded, and the engineering mind of Mark Albigence Waldo was primarily responsible for blazing the way for lifting the City of Bartow out of the class of sandy little villages into that of a modern Florida city.

He died of apoplexy on July 16, 1926, at his home in Bartow, and interment was made on July 20, at Manitowoc, Wis. The members of his immediate family surviving him are his widow, Mary Agnes (Ott) Waldo, to whom he was married at Jefferson City, Mo., on March 18, 1896; a daughter, Dorothy Wood Waldo, of Bartow; a son, Lieut. George Edward Waldo, U. S. A., of Fort Monroe, Va.; and a granddaughter. He is also survived by two sisters, Mrs. Edward H. Ross of Mansfield, Pa., and Miss Flora Waldo, of Manitowoc, Wis., and a brother, Mr. George E. Waldo, of Chicago, Ill.

Mr. Waldo was a Phi Kappa Psi at the University of Wisconsin; a Mason; a Knight Templar; a member of Egypt Temple, A. A. O. N. M. S., Tampa, Fla.; an active member of the Protestant Episcopal Church; a member of the Yacht and Country Club of Tampa, the Golf Club of Bartow, and the Engineers Club of New York, N. Y. He was a man who was quiet and unobtrusive in his views, always courteous, deeply interested in fishing and all outdoor sports, and one who made strong and steadfast friends. The Society has lost a member who was a credit to it; his friends have lost a companion whose place cannot be filled by another.

Mr. Waldo was elected a Member of the American Society of Civil Engineers on May 2, 1900.

JAMES HOLLIS WELLS, M. Am. Soc. C. E.*

DIED SEPTEMBER 24, 1926.

James Hollis Wells was born in England on August 3, 1864, the son of Arthur and Charlotte (Hollis) Wells. His grandfather, Richard Hollis, was an officer in the King's (1st) Dragoon Guards and fought under Wellington in the Battle of Waterloo. Later, he was Captain and Adjutant of the Grenadier Guards.

The family came to America when James Hollis Wells was still a boy and settled in Bethlehem, Pa. He received his education in the public schools of Bethlehem, and in his early manhood, taught in the Grammar Schools of South Bethlehem. He was graduated and received the degree of Civil Engineer from Lehigh University in 1885.

While attending the University, from 1883 until after his graduation, Mr. Wells was connected with the Borough Engineering Department of Bethlehem as Assistant Engineer, and with the Survey Department of the Philadelphia and Reading Railroad Company. From 1886 to 1889, he was with the Department of Public Works of New York, N. Y., as Assistant Engineer and Inspector of asphalt and other paving. From 1889 to 1891 he served as Engineer for J. D. and T. D. Crimmins, General Contractors, and in 1892 and 1893, he was Clerk of the Works for Cornelius Vanderbilt and Engineer for the firm of Renwick, Aspinwall and Russell, Architects, on its building work.

In 1893 Mr. Wells became associated with Mr. W. H. Russell. This partnership was afterward merged into the firm of Clinton and Russell, and, later, that of Wells, Holton and George, Architects. At the time of his death, Colonel Wells was the Senior Member of the firm.

He was well versed and quite active in various lines of engineering, and due to his close affiliation with the building industry was considered an expert in all questions of engineering, foundations, structural steel design, plumbing, heating, power-construction details, methods of building, and similar work. Among the prominent buildings with which he was identified, and in which unusual conditions were encountered, were the Hotel Astor, Hudson Terminal Buildings, Whitehall Building, Broad Exchange Building, Apthorp Apartments, Graham Court Apartments, Mutual Life Buildings, 7th and 71st Regiment Armories, and Mecca Temple in New York; Imperial Oil Building, Toronto, Ont., Canada; Humble Oil Building, Houston, Tex.; Whitney Central Bank Building, New Orleans, La.; Mutual Life Insurance Building, Cape Town, South Africa; and the Mutual Assurance Building, Life Insurance Company of Virginia Building, and National State and City Bank Building, Richmond, Va. In addition to his activities with his own firm, he was Advisory Architect of the Elks National Memorial Building, Chicago, Ill.

Colonel Wells was also appointed by Governor Woodrow Wilson of New Jersey in 1913 as a member of the New Jersey Bridge and Tunnel Commis-

* Memoir prepared by James Dougan, Assoc. M. Am. Soc. C. E.

sion, and served as Consulting Engineer of the United States Treasury Department from 1914 to 1918, on the foundations of the Treasury Building in New York, and the engineering and construction features of the Department of the Interior Building at Washington, D. C.

He played an important part in the business world of both New York and New Jersey. He was a Director of the Trust Company of New Jersey, a member of the Executive Committee and Chairman of the Auditing Committee of that Company. He was also President of the Realty Company of New Jersey, and a Director of the Colonial Life Insurance Company of America.

Colonel Wells inherited a love of military service from his distinguished grandfather and served for more than thirty years in the National Guard of New York. He began his military career in the Fifth Company, 7th Regiment, N. Y. N. G., in 1892, was commissioned Second Lieutenant, Company F, 71st Infantry, in December, 1892; Captain, in 1893; Major, in 1898; Lieutenant Colonel, in 1901; and Colonel, in 1917. He saw service with the 71st Regiment in Cuba as Major and commanded part of the regiment in the Battle of San Juan Hill on July 1, 1898. Later, he became Engineer Officer of the 1st Division, 5th Army Corps, in Cuba.

He was a member of the American Society of Mechanical Engineers; the Engineers' Club of New York; a member of the Board of Governors and Chairman of the House Committee of the Railroad Club of New York; member of the Carteret and Union League Clubs, of Jersey City, and the Westmoreland Club, of Richmond, Va.; Past-Master, Bunting Lodge, F. and A. M.; member of Mecca Temple, A. A. O. N. M. S., Scottish Rite Masons; Benevolent Protective Order of Elks, Jersey City Lodge No. 211, and many engineering and military societies. He had been Vice-President of the New York State Rifle Association and a Director of the National Rifle Association of the United States, and President of the National Guard Association of the State of New York. He was also Adjutant of the United States Rifle Team that won the Palma Trophy at Bisley, England, in 1903.

In April, 1892, Colonel Wells was married to Belle Porter White. They have one son, Thomas Richmond Wells. The family residence is in Jersey City, where Colonel Wells was prominent in social and civic affairs.

He was buried in New York Bay Cemetery, Jersey City. Among the honorary pall-bearers were former Governor John K. Tener, of Pennsylvania, Gen. William C. Heppenheimer, and Joseph T. Fanning, Past Grand Exalted Ruler of the Elks. Fifty-six officers from the 71st Regiment, New York, who were formerly under the command of Colonel Wells, attended the funeral services, as well as many representatives of the building industry.

Colonel Wells was elected a Member of the American Society of Civil Engineers on February 5, 1901.

FRANK THOMAS WESTCOTT, M. Am. Soc. C. E.*

DIED OCTOBER 22, 1924.

Frank Thomas Westcott was born at Burrillville, R. I., on August 6, 1868. His early education was obtained in the public schools at Milford, Conn., and he received his college training at Brown University, Providence, R. I., from which he was graduated in 1890 with the degree of Bachelor of Philosophy. In 1892, he was graduated from the Massachusetts Institute of Technology, at Cambridge, Mass., with the degree of Bachelor of Science.

During 1892 and 1893, Mr. Westcott held a position with the Triangulation Department of the Board of Survey, Boston, Mass., and in 1893 and 1894 he was Chief Assistant Engineer with C. A. Hicks, Engineer, of Needham, Mass., in which position his principal work consisted in land development.

From 1894 to 1896, he had charge of the outside work for the City Engineer's Office at Central Falls, R. I., which included sewer construction costing about $50 000, and also street construction.

In 1896, Mr. Westcott engaged in private practice as a Civil Engineer with an office in North Attleborough, Mass., which practice he continued until his death on October 22, 1924. From 1899 to 1907 he served as Superintendent of Highways and had charge of all municipal improvements for North Attleborough. In addition, he acted as Engineer for the town in connection with several proposed street railways and also for the Boston and Providence Interurban Railway. At this time he designed and constructed several miles of sewers for surface water and two highway bridges with steel spans and concrete floors.

In 1909, he acquired the business of the R. H. Tingley Company which he conducted in connection with the North Attleborough office. He also made studies for a storage reservoir of between 200 and 300 acres. From 1908 to 1924, Mr. Westcott held the position of Town Treasurer and was also a Member of the Finance Committee for the Town of North Attleborough.

Mr. Westcott was a capable, level-headed engineer, and a good citizen, quiet, efficient and hard-working, to whom many offices of importance were entrusted by his fellow townsmen.

On May 31, 1894, he was married to Elizabeth Nancy Tourtellot who, with a son, survives him.

Mr. Westcott was elected a Member of the American Society of Civil Engineers on January 3, 1911.

* Memoir compiled from information on file at the Headquarters of the Society.

JULIUS HERMAN GEORGE WOLF, M. Am. Soc. C. E.*

DIED DECEMBER 19, 1925.

Julius Herman George Wolf was born in Edwardsville, Ill., on January 6, 1874. He was the son of Otto and Elizabeth (Feisler) Wolf, the eldest of eight children. At the age of 15, he entered the University of Delaware from which he received the degrees of Bachelor of Civil Engineering in 1893 and Civil Engineer in 1896. He later completed his technical education at the Massachusetts Institute of Technology.

Mr. Wolf's earliest engineering experience was as a Construction Superintendent for an architect on the construction of cold storage plants and breweries in Boston, Mass., Cincinnati, Ohio, Buffalo, N. Y., and Lawrence, Mass. In 1896, he entered the service of the Metropolitan Water Board of Boston where he served as Inspector on the construction of the Wachusett Aqueduct. In 1898, he became connected with the United States Engineer Corps and for seven years served as Civilian Assistant Engineer on the construction of coast defense fortifications in San Francisco Harbor and in the Philippine Islands.

Beginning in 1905, and for about four years thereafter, Mr. Wolf was connected with mining in Nevada and California. In 1909 he became interested in the petroleum industry and was closely connected with oil interests until his death. Following his own recommendation for such survey, he was selected by the Secretary of the Interior in 1914 to direct the investigation of the character and extent of oil lands withdrawn from entry. In 1916, he was appointed Consulting Engineer to the United States Navy in connection with the investigation of the petroleum reserves of the Navy and the recovery of oil lands in the Valley Fields of California.

Following the completion of the Government litigation in the oil fields, and the passage of the Income Tax Act of 1918, Mr. Wolf carried on a consulting practice, principally in the appraisal of oil properties for income tax purposes. His engagements were with the principal oil companies of California, and his intimate knowledge of the geology of the California oil measures and all phases of the oil industry, supported by a very retentive memory, made him one of the best informed men on the subject in the profession. At the time of his death, in addition to his consulting practice, he was Managing Director of the Exploration Oil Company.

Mr. Wolf was married on April 3, 1907, to Blanche Cushman. He is survived by his widow, two sons, Wilsey and Norman Cushman and one daughter, Charlotte Cushman Wolf. His death occurred very suddenly on December 19, 1925, due to cerebral hemorrhage.

* Memoir prepared by Frank G. White, M. Am. Soc. C. E., and Dorsey Ash, Esq., San Francisco, Calif.

He was a member of the American Institute of Mining and Metallurgical Engineers, the American Petroleum Institute, and the Engineers Club of San Francisco. He had a large circle of friends and was regarded very highly, not only because of his sterling character and genial personality, but also because of his ability in his profession.

Mr. Wolf was elected an Associate Member of the American Society of Civil Engineers on May 7, 1902, and a Member on April 5, 1904. He was also a Member of the San Francisco Section of the Society.

HENRY BEECHER WOOD, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1926.

Henry Beecher Wood, the son of Charles Thomas Wood, of Bradford, Mass., and Sarah Jane (Harnden) Wood, of Portland, Me., was born in Woburn, Mass., on August 28, 1854. His ancestors, like those of his wife (the Eatons), were direct descendants of some of the first settlers from England, dating back to early Colonial times. His father was a Civil War Veteran and his grandfather a Veteran of the War of 1812.

Mr. Wood was educated in the public schools of Woburn. He was graduated from the Woburn High School and later entered the Massachusetts Institute of Technology, graduating in 1876 in the course of Civil Engineering. Immediately after his graduation he taught school in Woburn.

In 1880 he entered the employ of the Mississippi River Commission, United States War Department, on topographical work on the river below Memphis, Tenn. He later became Assistant Engineer on Hydrography and Topography, and from 1882 to 1885 was principally in charge of hydrography.

In 1885, Mr. Wood accepted a position with the City of Boston, Mass., on the Boston Water-Works, and was assigned to field work and computations on the construction of the Farm Pond Conduit. During the winter months he was engaged in the development of the Sudbury River water-shed, and on the survey of the Shawsheen water-shed.

From 1887 to 1891, he served as District Engineer in the Sewer Department of Boston, in charge of the design and construction of all sewers for about one-third of the city. He was also in charge of triangulation and topographical development of suburban territory.

In 1891, Mr. Wood was made Secretary and Executive Engineer of the Consolidated Street Department of Boston, which included what was formerly known as the Paving Department, Sewer Department, Bridge Department, and Sanitary and Street Cleaning Department, having charge also of improved sewerage, and smoke prevention. He held this position until 1897, when he was appointed Chief Engineer of the Massachusetts Topographical Survey Commission. He was in full charge of town, city, and State boundary surveys under this Commission and became an expert in triangulation work. His duties also included the preparation of an atlas of all boundaries, perambulation of lines, special river and pond surveys, etc.

On June 6, 1901, this work was transferred to the Board of Harbor and Land Commissioners, and under it Mr. Wood continued to be in full charge of his former duties until their completion on May 10, 1916, the final atlas of boundaries having been completed on February 11, 1916.

He also had full charge, in 1902, of the survey of the inland canal, known as the Brockton Canal, which extended from Weymouth, Fore River, through Brockton and Taunton to Fall River, Mass., as well as the Mount Tom Reservation Survey, Nashua River Survey, and other special work of that nature.

* Memoir prepared by John R. Freeman, Past-President, Am. Soc. C. E., and Charles T. Main, M. Am. Soc. C. E.

From July 20 to October 2, 1915, Mr. Wood's services were temporarily loaned to the Massachusetts Highway Commission and from May 10 to September 11, 1916, to the Bureau of Statistics to measure and complete the water areas in the State for census purposes.

Under the Division of Waterways and Public Lands, created on August 3, 1916, he continued as Assistant on general and research work. In 1918 he was engaged on the compilation and publication of a "Report on Water Resources of Massachusetts". On January 23, 1920, he was assigned to the care and custody of plans and records of the Department of Public Works, indexing, classifying, and regulating them in the Document Room at Commonwealth Pier No. 5.

After his retirement from the State service on August 29, 1924, Mr. Wood was engaged as Consulting Engineer on minor work for the Town of Belmont, Mass.

He presented a good summary of his principal State work in connection with the boundary surveys in a paper* read before the Boston Society of Civil Engineers on April 25, 1898, and published by that Society. In connection with the survey work it is interesting to note that Mr. Wood was continually experimenting with the design of stadia rods, both as to the most practical hieroglyphics and as to color. He was highly skilled in all phases of triangulation surveys and it is said by other engineers that no one outside the United States Coast and Geodetic Survey was more competent or skillful than Mr. Wood in this work.

In 1891 he became a member of Mount Horeb Lodge, A. F. and A. M., and, in 1912, of the Belmont Lodge at Belmont. He was also a member of the Canopy Club, Inc., of Boston, a Masonic fraternity including all State employees; of the Boston Society of Civil Engineers; and of the Massachusetts Highway Association. From December 13, 1893, to December 31, 1918, he was a member of the New England Water Works Association. He was also a member of the Appalachian Mountain Club from 1898 to 1914, and at one time a member of the Massachusetts Bicycle Club. At the time of his death Mr. Wood was a member of the Alumni Council of the Massachusetts Institute of Technology, representing the Class of 1876.

On March 21, 1883, he was married to Emily Wyman Eaton, who survives him. He is also survived by his sister, Annie Louise Wood, and two of his three children, Dana M. Wood, M. Am. Soc. C. E., and Mildred Louise Wood.

Mr. Wood was imbued with a quiet efficiency. He was a lover of his fellow men, and always carried with him a cheerful optimism that was a help and a joy to his friends. Nothing delighted him more than to hear a good anecdote and to relate one. He will be missed by all who knew him.

Mr. Wood was elected a Member of the American Society of Civil Engineers on April 3, 1895. He had also been a member of the Northeastern Section of the Society since its organization.

* "State, City, and Town Boundaries," *Proceedings*, Assoc. of Eng. Societies, Vol. XXI, 1898, p. 219.

CHARLES GRIFFITH YOUNG, M. Am. Soc. C. E.*

DIED JUNE 16, 1926.

Charles Griffith Young, the son of Charles Henry and Marion (Kellogg) Young, was born at Bath, N. Y., on November 1, 1866. He received his education at the Haverling Academy where he took special courses in technical subjects and from which he was graduated in 1885.

His first work was with the Schuyler Electric and Manufacturing Company, of Hartford and Middletown, Conn., with which Company, in 1886 and 1887, he held the position of Assistant Engineer of installation plants. During this engagement he also took an expert course in every department of the works. From 1888 to 1892, he served as Engineer and General Superintendent for the Mount Morris Electric Light Company, of New York, N. Y. His work at this time consisted of the construction of two central stations and one temporary plant, a complete distribution system underground and overhead.

From 1892 to 1900, except for a short period, Mr. Young acted as Supervising Engineer for the White-Crosby Company and the J. G. White Engineering Company, of New York, N. Y. In this position he made investigations and reports on electric lighting systems, power plants, steam railways, and electric railways in Baltimore, Md., Richmond, Va., Washington, D. C., Toledo, Ohio, Detroit, Mich., and in Haiti. He was also engaged on the construction of some of this work.

For a short time in 1898 and 1899 Mr. Young served as Supervising Engineer for the New York Heat, Light, and Power Company, his work including plant extension and operation. He also investigated and reported on the electric lighting situation in New York and on proposed steam railway, electric railway, and electric lighting installations in Porto Rico and in Havana, Cuba.

From 1900 to 1902 Mr. Young held the position of Engineer for J. G. White, Ltd., of London, England, and as such was engaged in investigations and reconnaissance work, design, and specifications for electric railways and lighting and power systems. During this period he estimated the cost and earnings of a complete electric railway equipment for Auckland, New Zealand. He also made estimates and investigated costs and earnings and made general plans for electric railways, lighting, and power plants at Para, Brazil; Montevideo, Uruguay; Buenos Aires and Rosario, Argentine Republic; and in the Philippine Islands.

He served as General Construction Manager from 1902 to 1909 for J. G. White and Company, in New York, on plant specifications, layout designs, and the construction of electric railways, power plants, and steam railways. He was also engaged on harbor and sea-wall contracts, dredging, water-works, estimates, analyses, and recommendations on the operation and equipment of properties. From 1892 to 1909, more than 2 000 miles of electric railways were built under his direction in the United States and foreign countries.

* Memoir compiled from information on file at the Headquarters of the Society.

From 1909 to December, 1917, Mr. Young carried on a general independent practice as a Consulting Engineer, with an office in New York. He specialized in investigations and reports of professional undertakings, both proposed and actual; as an expert before judiciaries and commissions on valuations, etc.; and on operating conditions of steam and electric railroads, water and rail terminals, hydro-electric power, etc.

In 1918, he served as Engineer for Ford, Bacon, and Davis, Incorporated, of New York, on contract negotiations, and, from 1919 to 1921, as Manager for John N. Willys Export Corporation, of New York, in which capacity he was engaged chiefly on export engineering problems.

In 1922, Mr. Young returned to private practice and organized the C. G. Young Company, Incorporated, of which he was President, which position he held at the time of his death. His engineering activities extended over a wide range, including Canada, South America, Siberia, and various European countries, in the course of which he had traveled three times around the world. He died suddenly in Porto Rico on June 16, 1926, while on a professional engagement.

On October 21, 1890, Mr. Young was married to Bessie Curtis Leonard, of New York, N. Y., who, with a daughter, Elizabeth D. Young, survives him.

He was the author of a valuable paper entitled "Logical Basis for Valuations", which was printed in pamphlet form and which also appeared in *Electric Traction* of January 21, 1911.

As an engineer Mr. Young was always courageous and forward-looking. In addition to practical engineering skill, he was distinguished by the happy faculty of always seeing beyond obstacles, and of viewing optimistically any work before him, with an inspired faith in its successful completion.

Those close to him knew him as an unselfish, devoted friend, whose loyalty ripened with the years. Even his casual acquaintances knew him as a man of high principle and fine character, and they remember him now for those lovable qualities that kept expressing themselves through his contagious smile.

He was a member of the American Institute of Electrical Engineers; National Electric Light Association; New York Electrical Society; American Electric Railway Transportation and Traffic Association; American Electric Railway Engineering Association; Pan American Society; Pan American Chamber of Commerce; Steuben Society; Engineers' Club of New York; New York Railroad Club; Indian House Club; and the Circumnavigators.

Mr. Young was elected an Affiliate of the American Society of Civil Engineers on March 6, 1894, and a Member on March 8, 1921.

ROSWELL JAMES AYDLOTTE, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 22, 1925

Roswell James Aydlotte, the son of John K. R. and Cora Aydlotte, was born at Chester, Pa., on March 9, 1892. He received his early education at the Chester High School and also at Bucknell University, Lewisburg, Pa. In September, 1910, he entered Drexel Institute, Philadelphia, Pa., where he took the Civil Engineering Course. He was graduated from that institution in June, 1914.

Mr. Aydlotte's engineering work began shortly after his graduation from Drexel Institute and led into many lines of the profession. In 1914-1915 he served as Surveyor in Class 1 of the United States Engineering Department, acting as Inspector of Construction, and in 1915-1916 he was engaged on the construction of sewers and highways in Oaklyn, N. J. In 1916-1917 he was employed on construction work at the Pennsylvania and New Jersey Shipbuilding Yards and in 1917 he was also engaged as Engineer in Hydraulic Dredging by the Delaware Dredging Company.

In 1917-1918 Mr. Aydlotte served as Chief Night Inspector of Ways and Piers at Hog Island, American International Shipbuilding Corporation and in 1918 his work took him to Boston, Mass., where he was Engineer for a contracting company on steam shovel work. During 1918-1919 he was employed as Engineer on the construction of a gravel-washing plant at Curtis Bay, Md., by the Arundel Shipbuilding Company.

In 1919 the Delaware River Dredging Company made use of his services in repairing the break in the sea-wall at Port Penn, Del., and in 1919 and 1920 he was a member of the firm of Aydlotte and Roop, Engineers and Surveyors, with offices in Philadelphia. In 1921 Mr. Aydlotte was appointed County Engineer of Delaware County, Pennsylvania, which position he occupied until the time of his death on February 22, 1925.

The most important phase of Mr. Aydlotte's engineering career began with his appointment as County Engineer of Delaware County. The office had been established only a short time before this, and Mr. Aydlotte was compelled to organize it on a large scale and get the force in condition for an active campaign of bridge replacement. The design of bridges in Delaware County had been done previously by engineers in private practice. Bridges had been constructed in a desultory fashion and the whole County was covered with structures that were never designed to carry the heavy loads imposed by modern traffic. Many of these bridges were dangerous either from overload or age. The situation was acute, and under Mr. Aydlotte's supervision the County Commissioners began the reconstruction of practically every bridge in the County.

It was in this work that Mr. Aydlotte showed his ability both as an engineer and as an executive. In the four short years from the time of his appointment as County Engineer until his death, he designed and super-

* Memoir prepared by C. M. Broomall, Assoc. M. Am. Soc. C. E.

intended the construction of approximately thirty-two bridges, mostly of the reinforced concrete slab or arch type. His last work was the design and supervision of the beautiful Memorial Bridge, a two-arch reinforced concrete span of imposing dimensions, on the Philadelphia and Baltimore Pike, over Crum Creek. This bridge was completed before Mr. Aydlotte's death, although he did not live to receive the tributes which were accorded to him at its dedication ceremonies. The responsibility and exacting work connected with the building of so many bridges in so short a time no doubt played a part in hastening his untimely death.

His pleasant personality endeared him to his many friends and his energy and ability as an engineer are attested by his record.

On December 15, 1915, Mr. Aydlotte was married to Edith Ward. One child, Margaret Jane Aydlotte, was born of this marriage.

Mr. Aydlotte was elected an Associate Member of the American Society of Civil Engineers on October 2, 1922.

RICHARD STOCKWELL BETTES, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 19, 1926.

Richard Stockwell Bettes, the son of Stockwell and Ellen (Cady) Bettes, was born in Springfield, Mass., on October 2, 1886. He was prepared for college in the public schools of that city, and received his technical education at Worcester Polytechnic Institute, Worcester, Mass., from which he was graduated in 1910 with the degree of Bachelor of Science in Civil Engineering.

He obtained an early training from his father, also a Civil Engineer, whose guidance and advice gave him a thorough foundation for his life's work. During summer vacations from 1907 to 1909 he worked as Instrumentman on surveys for G. N. Merrill and Company, of Springfield.

Following his graduation, in 1910, Mr. Bettes was employed as a Structural Draftsman in the Engineering Department of the American Bridge Company at Pencoyd, Pa.

From 1911 to 1913, he was with the Isthmian Canal Commission, on the construction of the Panama Canal, as Chief of Party on general surveys in the vicinity of Gatun Lake, and especially the detailed surveying, including lines and grades and laying out of work required for the construction of the Gatun Dam and Spillway. The two years spent in Panama, during which the very important and highly interesting construction of Gatun Dam and Gatun Locks was finished, gave Mr. Bettes an experience in engineering as applied to construction that was of great value.

From 1913 to 1915, he was employed part of the time as Mechanical Draftsman by the Toronto Power Company, Niagara Falls, Ont., Canada, on hydro-electric power development work; later, as Designing Draftsman for the Hooker Electro-Chemical Company, Niagara Falls, N. Y., on plant equipment; and, finally, as Structural Draftsman for the Atlas Portland Cement Company, of Northampton, Pa., on the design of reinforced concrete buildings.

From April, 1915, to February, 1916, Mr. Bettes served as Construction Engineer for the Pennsylvania Trojan Powder Company, of Allentown, Pa., in charge of the construction of nitric acid, ammonium nitrate, and sulfuric acid plants. From February to July, 1916, he was Designing Draftsman for the American Zinc Company, of Mascot, Tenn., on plant and equipment in connection with a lime disposal plant, and from September to December, 1916, he was Assistant Superintendent for the Raymond Concrete Pile Company, of New York, N. Y., on the pile foundations of a building erected for the General Chemical Company, at Marcus Hook, Pa.

During the World War, from September, 1917, to June, 1918, he served in the American Expeditionary Forces in France, with Company A, 29th

* Memoir prepared by H. W. Farrar, Esq., New York, N. Y.

Engineers, U. S. Army. As this experience undermined his health, he was sent to the U. S. Government Hospital, at Fort Bayard, N. Mex., where, however, he had charge of the erection of four hospital dormitories. He received his discharge from the Army and the hospital in December, 1918.

In June, 1919, Mr. Bettes entered the employ of the Strathmore Paper Company, at Mittineague, Mass., as Draftsman on general engineering work, from surveys to mechanical design, and also as Resident Engineer at its power plant, at Woronoco, Mass., supervising the construction of two 150 000-gal., cylindrical, reinforced concrete, fuel oil tanks, the design and construction of an oil pump house, and several houses for employees. In December, 1920, he resigned, due to the completion of the construction work. During 1921 and 1922 he was engaged in miscellaneous engineering and other work, including a position as Draftsman with the Fisk Rubber Company, at Chicopee Falls, Mass.

In November, 1922, Mr. Bettes returned to the Strathmore Paper Company as Supervising Engineer, both on plans and construction of a new three-arch, reinforced concrete highway bridge, 300 ft. long, across the Westfield River at Woronoco, Mass., representing both the Strathmore Paper Company and the Town of Russell, Mass., which were jointly interested in this project. The bridge having been practically completed in March, 1924, he was engaged in general engineering work for the Company until he resigned in the fall of 1924, on the completion of new construction.

From 1925 until his death, he served as Assistant Resident Engineer for the Massachusetts Mutual Life Insurance Company on its new buildings at Springfield, Mass., in charge of the accounting and detailed inspection of all work performed by the construction forces. The project consisted of a main office building, 400 ft. long by 300 ft. wide, four stories high, with a tower to the ninth floor, the exterior being of face brick and limestone trim, as well as an auxiliary building containing store house, power house, and garage, 60 ft. by 200 ft., two stories high, covering 22 acres, which was made into lawns, shrubbery, and tennis courts. This complete development involved an investment exceeding $4 000 000.

It will be noted from his record that Mr. Bettes applied himself strictly to an engineering career and at all times was deeply interested in any matters pertaining to his profession. He was a man of quiet, gentlemanly disposition, whose character and principles were of the highest order, dependable, and highly respected by all who worked or were associated with him. His untimely death has removed one who was distinctly a credit to his profession.

He was especially devoted to his home and family and took great pride, keen satisfaction, and enjoyment in giving serious thought and a large part of his time and energy to the proper up bringing of his family and to making his home life ideal.

Mr. Bettes was a member of the Springfield Lodge, A. F. and A. M., of Springfield, Mass., and the First Church of Longmeadow, Mass. He was also a member of the Engineering Society of Western Massachusetts and the Appalachian Mountain Club of Boston, Mass.

He was married on October 8, 1920, to Elfrida M. Johnson, of Holden, Mass., a niece of the late Phelps Johnson, M. Am. Soc. C. E. Besides his widow, he leaves two children, Clara Ellen and Richard Stockwell. He also leaves a sister, Mrs. Everett E. Thompson, of Springfield, Mass. At the time of his death he was a resident of Longmeadow, Mass.

Mr. Bettes was elected an Associate Member of the American Society of Civil Engineers on September 12, 1921.

ANSON WOOD BURCHARD, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 22, 1927.

In the death of Anson Wood Burchard, on January 22, 1927, the engineering industries of the United States lost a man of notable creative accomplishments.

Born at Hoosick Falls, N. Y., on April 21, 1865, Mr. Burchard received his earlier education in the public schools and then attended Stevens Institute of Technology, at Hoboken, N. J., from which he was graduated in 1885 with the degree of Mechanical Engineer. He was also the recipient in 1926 of the Honorary Degree of Doctor of Laws from Union College.

Mr. Burchard's early employment was along the line of his course at Stevens Institute of Technology, which prepares more particularly for mechanical work. His first fifteen years of experience were in factory engineering and in manufacturing establishments at Danbury, Conn.

Two years (1900 to 1902) of experience as Vice-President of the Cananea Consolidated Copper Company and of the Greene Consolidated Copper Company led to his entering the employ of the General Electric Company, of which he became Comptroller, with headquarters at Schenectady, N. Y. His creative work in conjunction with the President of the General Electric Company, in the successful accomplishment (1902 to 1905) of a consolidation of the Fort Wayne Electric Company, the Stanley Electric Manufacturing Company, the General Incandescent Arc Light Company, the Northern Electric Company, and the Sprague Electric Company as units of the General Electric Company was an achievement of the highest order. He thus was instrumental in aiding to develop a great manufacturing organization with financial strength sufficient to enable it to carry out designs and spend money in research far beyond the resources of any of its individual component companies.

His career is another illustration of the manner in which graduates from engineering schools have capably grasped the problems of finance and administration in the engineering industries. Mr. Burchard's advancement with the General Electric Company was through the post of Assistant to the President, Vice-President, and member of the Board of Directors. In 1922 he was elected Vice-Chairman of the Board of Directors and Chairman of the Executive Committee of the General Electric Company, and, in the same year, President and Chairman of the Board of the International General Electric Company. The Presidency of the latter Company he relinquished subsequently, but continued as Chairman of the Board.

During the World War, Mr. Burchard volunteered as Assistant to the Secretary of War and served from January, 1918, until March, 1919, giving particular attention to the construction and development in the United States of increased facilities for the production of munitions of war.

* Memoir prepared by H. de B. Parsons and Dugald C. Jackson, Members, Am. Soc. C. E.

His personal characteristics and wise judgment in finance and administration, as related to the engineering industries, led to his becoming a constantly sought counsellor in the development of public utility facilities both at home and abroad. He had a master mind for formulating policies in developing the financial resources and credit of public utilities, thus enabling them to increase their buying power, and for expanding the importance of central station work, which resulted in stimulating the electric industry throughout the world.

Mr. Burchard's sound judgment led to his selection as a Director or Trustee of many organizations, namely, American Gas and Electric Company; American Power and Light Company; Andersen, Meyer and Company; Asheville Power and Light Company; American and Foreign Power Company, Incorporated; Barcelona Traction, Light and Power Company; California Electric Generating Company; Carolina Power and Light Company; Central States Electric Corporation; Electric Investment Corporation; Electric Investors, Incorporated; Electric Power and Light Corporation; Electric Railway Equipment Securities Corporation; Electrical Securities Corporation; General Electric Company; International General Electric Company; International Power Securities Corporation; Lehigh Power Securities Corporation; Mercantile Safe Deposit Company; Montana Power Company; Mohawk-Hudson Power Corporation; North American Company; New England Power Association; Northeastern Power Corporation; Power Securities Corporation; Republic Railway and Light Company; United States and Foreign Power Securities Corporation; Utah Securities Corporation; United Electric Securities Corporation; Western Power Corporation; Worthington Pump and Machinery Corporation; Yadkin River Power Corporation; The British Thomson-Houston Company, Limited; Compagnie des Lampes; Compagnie Française Thomson-Houston; Compagnie Generale di Elettricita; Société d' Électricité et de Mécanique; Sociedad Iberica de Construcciones Electricas; China General Edison Company, Incorporated; Société Edison Clerici Fabbrica Lampade; Société Financiere pour le Développement de l' Électricité; and Nippon Denki Shoken Kabushiku Kaisha (Japanese Electric Bond and Share Company).

Mr. Burchard differed from many other men because as he became older and more stressed by business details, he took upon himself outside personal work. As years passed, he undertook to help others in an increasing degree. He made time for interviews and was always helpful in giving assistance, to the end that the outsider's problem became his problem. This made him admired by all, and he was personally liked not only by his friends, but by his employees.

He had a knowledge of and confidence in fundamentals, whether of scientific, financial, or business character, and the result was clear thinking and correct application of efforts to achieve an end. Similarly, he was possessed of that peculiar faculty of intuition whereby he could reach into a subject-matter and at once see the facts in their correct relationship. It was due to this faculty that much of his success was founded.

He also had the ability to handle men having divergent views, and to get them together in agreement and accord. His interest in the development of science as applied in manufacturing, and his interest in education, are indicated by his membership in the Charles A. Coffin Foundation Committee and his Trusteeship of Stevens Institute of Technology.

Mr. Burchard was a member of various engineering societies, including the Iron and Steel Institute of Great Britain, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers.

His sociability and regard for his fellow man are indicated by the list of his clubs, which included the Metropolitan, of Washington, D. C., and Metropolitan, University, Engineers', Racquet and Tennis, Automobile, Westchester Country, Recess, City-Midday, Bankers, Piping Rock, and Nassau Country, in the New York Metropolitan District.

He was married in London, England, on December 5, 1912, to Allene (Tew) Hostetter, daughter of Charles H. Tew, a retired banker of Jamestown, N. Y.

His death occurred suddenly of acute indigestion, and came as a shock to all his associates. Tributes expressed by his friends and colleagues came from over the world. A few sentences from Mr. Owen D. Young give their general consensus:

"Due to his long association with the Company [The General Electric Company], Mr. Burchard had become familiar with its business in all its branches. His loyalty to the Company and his pride in it, his devotion to his associates, and his wide interest in all good causes, made him universally respected and loved. His loss to all of us is deeply felt."

Mr. Burchard was elected an Associate Member of the American Society of Civil Engineers on May 3, 1893.

THOMAS PENGELLY ELLIS, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 27, 1920.

Thomas Pengelly Ellis was born at San José, Calif., on October 22, 1884, but received his preliminary education in Canada. Removing with his father to Denver, Colo., he entered the Colorado School of Mines, from which he was graduated in 1907 with the degree of Engineer of Mines.

While attending the School of Mines, Mr. Ellis was engaged as Field and Office Assistant in the Engineer's Office of the Denver Tramway Company on the design and layout of electric sub-stations, curves, and switches. He was also engaged as Transitman and Assayer in the mines of the Cripple Creek and Clear Creek Districts, Colorado.

After his graduation, Mr. Ellis spent a year in the State of Washington on the sub-division of a portion of Mercer Island, Lake Washington, and as Contractor's Engineer in re-grading work in Seattle. After a few months of appraisal work for the Denver Union Water Company, Denver, and in mining and geological work in Clear Creek County, in March, 1908, he entered the employ of the Bureau of Engineering of the City and County of Denver, as Draftsman on the design and assessments of street improvements, sewers, etc. In March, 1910, he was appointed Inspector of Construction on part of the sewerage system, and in October of that year was made Division Engineer in charge of highway improvements, which position he retained until April, 1911. The occasion of his retirement from the Bureau of Engineering on a question of ethics was greatly to his credit.

In 1911, Mr. Ellis removed to San Diego, Calif., and until December, 1917, was connected with the Volcan Land and Water Company and the Cuyamaca Water Company, as Office Engineer, Field Engineer, Engineer of Land Department, and Valuation Engineer.

From February, 1918, to April, 1920, he was engaged as Resident Engineer for the Southern Division of Shipyard Plants of the United States Shipping Board as Supervisor and Inspector.

In April, 1920, Mr. Ellis became a partner of Mr. D. A. Dickie in the Dickie Construction Company and was engaged in salvaging piling at the Liberty Plant at Alameda, Calif. In the midst of these activities, he died from pneumonia on September 27, 1920. He is survived by his widow and two children.

Mr. Ellis was a man of great charm, high technical ability, and held steadfastly throughout his career to the ethics of the Engineering Profession. In his young manhood he had served as an artilleryman in the Canadian Militia and retained throughout his life an alert military carriage. Much to his regret the development of heart trouble prevented him from offering his services to his country in the World War. As a citizen of San Diego, he was highly thought of by the community.

Mr. Ellis was elected an Associate Member of the American Society of Civil Engineers on May 13, 1918.

* Memoir prepared by William S. Post, Assoc. M. Am. Soc. C. E.

BERNARD HOOE FOWLE, Jr., Assoc. M. Am. Soc. C. E.*

DIED APRIL 23, 1926.

Bernard Hooe Fowle, Jr., was born in Spokane, Wash., on August 10, 1890. He was the eldest son of Bernard Hooe and Milly Dorsey Fowle. Through his father he was related by blood to the Washington, Ball, Chichester, Hooe, and Briscoe families, and was directly descended through his mother from George Mason, of Gunston, the author of "The Bill of Rights", and from Chief Justices Chew, of Pennsylvania, and Dorsey, of Maryland. He was related to the Dallas and Bush families, of Pennsylvania, and the Carroll, Murray, and Howard families, of Maryland.

His first ten years were spent in Spokane. He then moved with his family to Washington, D. C., where he attended private schools, entering Virginia Polytechnic Institute, Blacksburg, Va., in 1909. Mr. Fowle's four years at the Institute were distinguished by high scholastic standing. He was graduated as First Lieutenant and Adjutant in June, 1913, receiving a letter of thanks for his fearless and efficient performance of duty as a cadet officer, his actions having manifested a high order of moral courage.

On graduation Mr. Fowle became a Draftsman for the Clinchfield Coal Corporation of West Virginia. In March, 1914, he transferred his activities to highway construction in Mercer County, West Virginia, as Resident Engineer. In November, 1915, he became Engineer for Mr. John L. Vaughan, at Petersburg, Va., on various development projects and electric interurban railway work. In May, 1916, he entered the service of the Virginian Railway Company as Instrumentman and later was promoted to the position of Resident Engineer.

In May, 1918, Mr. Fowle left the service of the Virginian Railway Company to take charge of a residency for the Chesapeake and Ohio Railway Company. He entered the Army as Second Lieutenant of Engineers in July, 1918. As such, he served as Company Commander of "C" Company, 70th Battalion of Engineers, Fort Douglas, Utah, and, later, as Topographical Officer. The Battalion was ordered overseas, but reached the port of embarkation on Armistice Day.

In December, 1918, he was honorably discharged from the Army and returned to Utah where he served with the State Road Commission as Locating Engineer and, later, as District Engineer of the Southeastern and Southwestern Districts. In September, 1920, he returned to the service of the Virginian Railway Company, acting as Resident Engineer on several construction projects and, later, as Assistant Engineer with the Valuation Department. A slipshod piece of work by Mr. Fowle was unknown.

It should be added that Mr. Fowle was hard of hearing. The reason for his acceptance by the Army has never been explained, but it was doubtless due to a persistence and determination that considered no discouragement nor took into account repeated refusals. Such a handicap to any man expecting to see active service at the front required a degree of courage

* Memoir prepared by R. M. Miller, M. Am. Soc. C. E.

possessed by few. To him the difficult part was that played by the Medical Corps and not that played by the enemy.

Bred in the fiber of this man was the knowledge of the amenities drawn from a long line of high-minded people. The practice of the courtesies was to him as easy and natural as breathing. Those who knew him in war times and in civil life have learned much of those things that make of life something very much worth while.

Endowed with a brilliant mind that had the ability to grasp quickly and retain in usable form impressions gained from inspection or study, Mr. Fowle's career was, indeed, cut far too short, but the extent of the influence of such a life is without measure. Endowed also with high moral and physical courage, without a flaw, it can be said of him, "verily, he was a man."

On March 12, 1921, he was married to Lucille White, daughter of Mr. and Mrs. J. W. White, of Princeton, W. Va., who, with two children, survives him.

Mr. Fowle was elected an Associate Member of the American Society of Civil Engineers on July 6, 1920.

CLARENCE STEPHENS GALE, Assoc. M. Am. Soc. C. E.*

DIED JULY 3, 1926.

Clarence Stephens Gale, the son of William R. and Annie (Stephens) Gale, was born on November 1, 1885, in Kent County, Maryland. His early education was acquired in the public schools of Kent County and Baltimore, Md.

In 1906 Mr. Gale entered the employ of the City of Baltimore as Assistant Engineer. Subsequently, he assumed the duties of Resident Engineer for the Maryland State Highway Commission, and advanced rapidly under W. W. Crosby and H. G. Shirley, Members, Am. Soc. C. E. At the time of Mr. Gale's resignation he was holding one of the most responsible positions in the Department as a result of his ambitious nature and close attention to work. He resigned from the State Highway Commission to assume the duties of Engineer for the Portland Cement Association, a position which he held with great credit. During the World War he had charge of road construction at Camp Meade and the Aberdeen Proving Grounds.

Mr. Gale resigned from the Portland Cement Association to become Construction Engineer of the Delaware Highway Department, and held this position with his usual efficiency until he resigned to become Engineer-Superintendent of the Fisher-Carozza Company, of Baltimore. With this firm, he made an excellent record for completion of work under the adverse conditions existing during the war period.

In 1920, Mr. Gale became associated with the Highway Engineering and Construction Company of Delaware, and the many millions of dollars worth of work that he completed are a tribute to his executive ability. During this period, he evolved several labor-saving devices and became a leader among large highway contractors in the amount of work completed. He made many records of progress that as yet have been unexcelled.

Later, he accepted a position as Engineer-Superintendent for the North Carolina State Highway Commission on special and difficult work. This appointment was made on account of his exceptionally fine record for speed and economy. He had resigned a few days prior to his death to accept the position of Engineer-Superintendent for the large highway contracts of James O. Heyworth, Incorporated. He died on July 3, 1926, after a brief illness.

His excellent record stands out as symbolic of the man who gave his entire thought and effort to his daily work. Through Mr. Gale's magnetic personality he quickly made friends whom, through his honest and upright dealing, he retained, and they honor the memory of one who gave his untiring efforts to his work, his friends, and his family.

He was married on December 30, 1914, to Martha Griffith, daughter of Mr. and Mrs. C. F. Griffith, of Easton, Md., and is survived by his wife and one son, Clarence Stephens, Jr.

Mr. Gale was elected an Associate Member of the American Society of Civil Engineers on June 11, 1917.

* Memoir prepared by C. L. Mann and Charles M. Upham, Members, Am. Soc. C. E.

WILLIAM HANNUM, Assoc. M. Am. Soc. C. E.*

DIED MAY 8, 1925.

William Hannum, the son of Preston E. and Charlotte E. (Philips) Hannum, was born on March 26, 1890, at Christiana, Pa. He received his early education in the schools of that town, and was graduated from the High School in 1906.

During the summer months of 1906, Mr. Hannum made his first acquaintance with engineering work as Rodman with an engineering corps on the construction of an earth embankment reservoir at Blairsville Intersection, Pa. From the very first his ability to handle men was evident, for at odd times he acted as Foreman over a gang of laborers and before long became the regular Labor Foreman.

In September of that year, however, Mr. Hannum decided that an engineering education would be an advantage if he expected to advance in this particular line of endeavor; so he left his position for two years of preparatory work at the West Chester State Normal School. The vacation months of 1907 found him back with the same Superintendent, as Labor Foreman and Team Boss, on a masonry reservoir at Johnstown, Pa., for the American Pipe Manufacturing Company, of Philadelphia, Pa.

The following four years were spent at Pennsylvania State College, from which Mr. Hannum was graduated with honors in 1912. As with most students, he spent his vacations at whatever work came to hand. One summer he was employed on a cattle steamer to England, where he spent two months in sight-seeing; another vacation was spent as a laborer in a lumber mill in Pennsylvania; and still another in the grain fields of the Middle West, and also as a clerk in a hotel at a fashionable summer resort in Pennsylvania.

After graduating in 1912, Mr. Hannum obtained a position as Engineer with the J. G. White Engineering Corporation, and was sent to Parkesville, Tenn., on a hydro-electric development. Although rated as an Engineer, he was made Material Tracer, and for eight months traveled through the Southern States, tracing shipments of construction material. This work, however, was shut down before completion, and Mr. Hannum was forced to seek other employment.

In August, 1913, he obtained a position as Labor Foreman with Latta and Terry, Contractors, of Philadelphia, on the foundations of the elevated railway escalator at Delaware and Market Streets, Philadelphia. Later, he was in charge of the construction of a sand and gravel washing plant at Bridgeboro, N. J., for the same firm.

The stringent times of January, 1914, found Mr. Hannum out of work. After a short time he accepted a position as Bond Salesman for N. W. Halsey, of Philadelphia. In this line of endeavor he proved to be very successful, but since engineering work had the greatest appeal, he resigned on September 1,

* Memoir prepared by H. G. MacMurchy, Constr. Supt., Phœnix Utility Co., Memphis, Tenn.

1916, to form a partnership under the name of McCool and Hannum, Contractors and Engineers. This partnership consisted of Messrs. Paul H. Turnbull, John McCool, and William Hannum, with offices in Philadelphia. The engineering operations were confined to that vicinity and consisted of building sewers for the City of Philadelphia; gas lines for the Suburban Gas and Electric Company; and several buildings of moderate size. The entrance of the United States into the World War, however, with the attendant rise in costs of labor and material, found the new firm unable to weather the financial storm and, in June, 1917, the partnership was dissolved.

On October 1, 1917, Mr. Hannum returned to the J. G. White Engineering Corporation, and was sent to U. S. Nitrate Plant No. 1, this work having just been started at Sheffield, Ala. The first problem was to organize a construction crew and Mr. Hannum, as Assistant Superintendent, was detailed to this task. He also acted as Special Purchasing Agent of Construction Plant and Equipment, later organizing the Purchasing Department. In May, 1918, he became Division Superintendent of Construction, taking charge of the construction of houses, school buildings, streets, sidewalks, sewers, sewage disposal plant, water supply system, water filtration plant, and other activities in the permanent village for the operating employees of the plant. This work was completed in the summer of 1919, but Mr. Hannum remained in charge of the force that closed down operations, returning to the New York Office in November, 1919.

His next assignments for the J. G. White Engineering Corporation, consisted of a one-story brick and steel warehouse at Pittsburgh, Pa., for the General Chemical Company; a wireless telegraph receiving station at Lakewood, N. J., and, later, extensions to a pulp mill at Chandler, Que., Canada. When this last work was completed in June, 1921, the ensuing business depression found Mr. Hannum out of a position. He, however, became connected temporarily with Rogers and Hagerty, Incorporated, of New York, N. Y., which firm was working on the Hill-to-Hill Bridge at Bethlehem, Pa. As this work was not in line with his ability, he resigned in April, 1922, to take a position with the Utilities North American Corporation, of Buffalo, N. Y., to sell stock. This proved to be a bad venture and in February, 1923, Mr. Hannum returned to his old connection with the J. G. White Engineering Corporation, and was sent to Ronceverte, W. Va., on the construction of a steam power station and the installation of equipment for the Virginia Western Power Company. The next assignment was the construction of the village for the operating employees of the East Pennsylvania Electric Company at Pine Grove, Pa. This work was completed in February, 1924.

In April, 1924, Mr. Hannum resigned his position with the J. G. White Engineering Corporation to accept employment as Construction Superintendent with the Phœnix Utility Company, and was sent to Memphis, Tenn. The work was the completion of a steam electric station for the Memphis Power and Light Company, and consisted of the installation of a 20 000-kw. turbo-generator, with its auxiliaries; also the moving of two smaller units from another station to be used for stand-by service; the building of two sub-

stations in Memphis; and also similar plant construction work in Jackson, Tenn., and Little Rock, Ark.

It was while in the midst of this program that Mr. Hannum was called to an untimely death. The Engineers Club of Memphis called a Convention of the Engineers of the Mid-South for May 6 to 8, 1925, and Mr. Hannum, as a member of the Sight-Seeing Trip Committee, assisted in entertaining the visitors, opening the Power Plant and its construction work for their inspection. After the tour of the industries of the city, those attending the Convention were taken aboard two Government boats, the *Choctaw* and *Norman,* and carried down the Mississippi River about 25 miles to inspect some revetment work that was being done by the Government Engineers. The trip down the river to Cow Island Bend was delightful and the inspection of the willow mats very interesting.

Before starting on the return trip, word was passed around that all civil engineers would transfer to the *Norman,* to discuss the formation in Memphis of a Local Section of the American Society of Civil Engineers. Mr. Hannum had already returned to the *Choctaw* with several friends, and was comfortably seated on the upper deck. A fellow engineer called for him and Mr. Hannum transferred to the *Norman,* so as to be present at the conference and join in the discussion.

The *Choctaw* took the lead on the return trip and, being a faster boat, in a little more than an hour had passed out of sight, hidden by the bends in the river. By the time the conference was in full swing on the upper deck of the *Norman,* it was noticed that the boat was listing slightly and a few minutes later the pilot sent orders for the passengers to move to the side of the boat. Shortly after, a second request was made for the passengers to shift positions, only to be followed in a few minutes by a decided list to starboard, which continued slowly until the boat had turned completely over. Mr. Hannum, with many others, climbed aboard the up-turned boat and rendered such assistance as he could to those in a more perilous position. It was at this time that he performed a noble deed that is best described by Alfred M. Lund, M. Am. Soc. C. E., of Little Rock, Ark., an eye-witness:

"When the wooden superstructure of the *Norman* came up and put the boat on its side it brought to the surface the body of a woman. Mr. Hannum immediately jumped from the hull of the boat down into the water, which was still over the side of the boat, and calling for help endeavored to drag the woman's body to a position which at that time appeared to be safer. I was about 10 ft. down stream and seeing he was unable to carry her by himself I went to his assistance. He had secured the body by the shoulders and was half lifting and half dragging it to the exposed part of the hull when suddenly the boat went down and that was the last I saw of them."

In describing the incident the *Commercial Appeal* fitly claimed that, "he died a hero". Eighteen days later, Mr. Hannum's body was found fifteen miles below the scene of the disaster. The body was taken to Florence, Ala., for interment.

A disaster that removed twelve engineers, five women of their party, a boy, and five members of the crew, was a sad blow to the Engineering Fraternity of the Mid-South.

Beloved by his associates and friends, Mr. Hannum will be sadly missed and his death leaves a gap that will be hard to fill. He was a Tau Beta Pi of Beta Chapter of Pennsylvania State College; a member of the Engineers' Club of Memphis, and a member of Zamora Temple, A. A. O. N. M. S., of Birmingham, Ala.

Mr. Hannum was married on September 14, 1919, to Annie Lipscomb, of Florence, Ala., who, with a daughter, Charlotte, three years old, survives him.

Mr. Hannum was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919.

DAHYABHAI BALABHAI KORA, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 22, 1926.

Dahyabhai Balabhai Kora was born of a noble family of Jain Baniss, in Kaira, now Jamnagar, Bombay, India, on September 26, 1886. After receiving his early education in Gondal, India, he was graduated in 1908 from the Poona College where he received scholarships every year. He received the degree of Licensed Civil Engineer from the University of Bombay.

In 1909 and 1910, he served with the Public Works Department of the State of Baroda, India, on building construction, roads, sanitation, drainage, etc.

From 1910 to 1917, Mr. Kora was engaged as Assistant Engineer of the State of Gondal, India, on the design and construction of buildings, roads, bridges, irrigation, water-works, town improvement, gas and telephone works. During this period he designed and constructed 50 miles of district roads, including highway, arch, and skew bridges. He also made a survey and design for a causeway across the River Vinu and for a drainage project for a part of the City of Gondal. His work also included an impounding reservoir for a combined irrigation and water supply works and the preliminary survey and design for an irrigation plan at Gomta, as well as an irrigation project with an impounding reservoir of earthen dams and masonry waste weir to irrigate about 3 000 acres. In addition, Mr. Kora designed and constructed many public buildings for the State of Gondal, including schools, museums, public offices, etc., using the purely Hindu style of architecture.

From 1911 to 1918, in addition to other duties, he acted as Honorary Examiner in Architecture and Civil Engineering at the Technical Institute, State of Baroda, except for two years when he was Officer in Charge of the Gondal State Camp, Delhi Durbar.

In 1917, Mr. Kora was engaged as Temporary Engineer for the Government of the Central Provinces, India, in charge of the Chhindwara District, on the maintenance and repair of 200 miles of district roads, the construction of a new road, 12 miles long, bonded warehouses, bridges, etc.

In 1918, he was appointed Engineer for Irrigation and Communications, which position he held until September, 1918, when he was made State Engineer of Nawanagar, India, in charge of the Public Works Department and Irrigation. As such, he was engaged on the design and construction of important public buildings, roads, bridges, harbor, irrigation, sanitary, and reclamation works, costing in all more than £1 000 000 sterling. He constructed twelve masonry dams for irrigation across catchment areas from 5 to 200 sq. miles in size and two bunds to reclaim about 5 000 acres of land, as well as approximately 60 miles of district roads and two raised causeways across a river having a catchment basin of 125 sq. miles. Many of the fine public and State buildings by which the City of Jamnagar has attained the

* Memoir compiled from information on file at the Headquarters of the Society.

reputation of being the most beautiful in Kathiawar, were designed and erected under Mr. Kora's careful supervision.

Mr. Kora was an able engineer and a skillfull designer who carried out his work with a high sense of duty, honesty, and straightforwardness. In his private as well as in his public life he endeared himself to all by his genial disposition, scrupulous honesty, and conscientious discharge of his various duties. His character was exemplary and his manners were simple and unassuming.

The State and the public expressed their deep sorrow at Mr. Kora's death by closing all offices, schools, and shops, in his honor. An Extraordinary Gazette of the State was also published, eulogizing the numerous services rendered by him, and public meetings were held at various places expressing deep sympathy with his parents and family.

In spite of his arduous responsibilities he kept in touch with the progress of engineering science in foreign countries. He was a member of various technical societies, including the Society of Engineers; Institute of Structural Engineers; Institute of Municipal and County Engineers; Royal Sanitary Institute; and the Institute of Indian Engineers. He was also associated with the American Institute of Architects and the Royal Institute of British Architects, and had contributed important papers at various engineering conferences.

Mr. Kora was elected an Associate Member of the American Society of Civil Engineers on August 9, 1920.

HAROLD CHANDOS LYONS, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 30, 1926.

Harold Chandos Lyons was born in New York, N. Y., on January 7, 1883, the son of Wallace Foster and Clara Louise (Cornwell) Lyons. He was educated in the public schools of New York, later attending Phillips Academy at Andover, Mass. From Andover he went to Princeton University in 1901, and was graduated from the School of Science in 1907, with the degree of Civil Engineer.

Mr. Lyons began his engineering work in 1904, when he served as Chainman and Instrumentman of the Pennsylvania East River Tunnel, in New York. From 1907 to 1909, he was Engineer in Charge and Superintendent of Construction of four real estate developments—Queens County, Terminal Heights, Elmhurst Square, and Kissena Park. In 1910, he became Superintendent of Construction of Forest Hills Gardens, Long Island, a model town built by the Russell Sage Foundation. His work included buildings, roads, sewers, water-works, landscaping, and various other kinds of work involved in real estate development. He was the first Superintendent of this development, organizing the personnel and instituting the cost observation and keeping methods.

From 1911 to March, 1913, Mr. Lyons was Principal Assistant Engineer of the Construction Service Company, of New York. At this time he was engaged in investigating and recording methods and costs of engineering and contracting enterprises.

Until the entrance of the United States into the World War, he was engaged in the Contracting and Engineering Professions, in the firms of Lyons and Doyle, Incorporated, and Page and Lyons. He was considered an authority on building construction economics.

Mr. Lyons entered the Army shortly after the declaration of war by the United States, and was a First Lieutenant in the Engineer Section of the Officers' Reserve Corps. In November, 1917, he went overseas with the 20th Engineers, and was engaged in various branches of engineering work in France. He was promoted to the rank of Captain in August, 1918. For exceptionally meritorious and conspicuous services at Tours, he received a citation from General Pershing. In August, 1919, he was returned to the United States and was placed on temporary duty in the Office of the Chief of Engineers, later being honorably discharged from his temporary commission.

From 1919 to 1920, Captain Lyons served as Senior Highway Engineer of the Bureau of Public Roads, United States Department of Agriculture. On July 1, 1920, he was commissioned a Captain, Corps of Engineers, U. S. Army, and was stationed at posts in Texas and Virginia until September, 1922, when he entered the Engineer School at Fort Humphreys, Virginia. On completion of that course, he was on duty as Topographical Officer at Camp Meade, Maryland, and, later, was detailed as Assistant Professor of Military Science

* Memoir prepared by L. R. Lohr, Assoc. M. Am. Soc. C. E.

and Tactics at the Carnegie Institute of Technology, Pittsburgh, Pa., which position he held until his death.

The death of Captain Lyons came as a distinct shock to his many friends and associates in his work. He is recalled most pleasantly by all those who came in contact with him, and it was with great regret that they learned of his death, which occurred at the Walter Reed General Hospital, Washington, D. C., on December 30, 1926.

He will always be remembered with the highest esteem, not only for his unusual ability in the field of engineering and his gallantry as an Army leader, but for those rare personal qualities which charmed all who knew him. The following quotation from the Carnegie School paper shows the position which he held in the hearts and minds of those with whom he worked:

"Captain Lyons was one of those few individuals possessing a personality, character, and ideals that compel the deep admiration of all who come in contact with him. Because of his unceasing efforts, the standards of the Reserve Officers' Training Corps at Carnegie have been raised immeasurably."

He was married to Esther Gibson, daughter of Mr. and Mrs. James Gordon Gibson, of San Antonio, Tex., in 1922. Captain Lyons is survived by his widow and two nieces, Mrs. Gerard Blackburn, of Bombay, India, and Miss Jean Slater, of Yonkers, N. Y.

A member of several organizations, he was active in all of them. He was a member of the Army and Navy Clubs of New York and Washington; the Society of American Military Engineers; the Engineers' Society of Western Pennsylvania; the American Legion Liberty Post; Veterans of Foreign Wars; Americus Lodge No. 535, F. and A. M., New York; Syria Mosque, A. A. O. N. M. S., Pittsburgh; the Sojourners' Club; Acacia Fraternity, and the Scabbard and Blade, Carnegie Chapter. He was a greatly respected member of these organizations.

Captain Lyons was elected an Associate Member of the American Society of Civil Engineers on July 9, 1912.

HARRY ADOLPH OESTERLE, JR., Assoc. M. Am. Soc. C. E.*

DIED MAY 24, 1926.

Harry Adolph Oesterle, Jr., the son of Henry and Margaret (Weber) Oesterle, was born in Newark, N. J., on December 23, 1891. Following his primary education he entered the Newark Technical School where he studied for four years, supplementing this work by special courses at the Brooklyn Polytechnic Institute, Brooklyn, N. Y.

During 1913, Mr. Oesterle was connected with the Research Corporation of New York, N. Y., on the design of electrical precipitators and, later, with the Guarantee Construction Company on general plant construction and ash-handling systems.

From June, 1914, until March, 1915, he held the position of Designer with The Balbach Smelting and Refining Company, of Newark, N. J., on the design and construction of an electrical precipitator for smelter flue gases.

In March, 1915, Mr. Oesterle accepted a position with The Westinghouse, Church, Kerr, and Company, of New York, as Designing Draftsman and, later, was promoted to the position of Junior Engineer. His work consisted principally of power-house and sugar refinery design.

From December, 1916, until his death he served as Resident Engineer of the New Niquero Sugar Company, of Central Niquero, Cuba, in complete charge of all field work, including railroad construction and maintenance, as well as surveying and general design and construction. It was while employed by the New Niquero Sugar Company that Mr. Oesterle met his death as the result of a premature dynamite explosion.

He was married on March 14, 1918, to Sarah T. Maher, of Newark, N. J., who, with a daughter, Sarah Adelaide, survives him.

He was an Associate Member of the American Society of Mechanical Engineers and a Member of the American National Engineering Societies of Cuba.

Mr. Oesterle was elected an Associate Member of the American Society of Civil Engineers on July 6, 1925.

* Memoir compiled from information on file at the Headquarters of the Society.

DAVID THOMAS PITKETHLY, Assoc. M. Am. Soc. C. E.*

DIED APRIL 21, 1926.

David Thomas Pitkethly was born in Boston, Mass., on March 29, 1876. In the fall of 1890 he entered the employ of Wheeler and Parks, Consulting Engineers, of Boston, and served as office boy and Assistant on drafting and surveying until 1894. In 1895, he was with George W. Stadley and Company, Map Publishers, Boston, on surveys for atlases, and, in 1896 and 1897, he held the position of Draftsman in the Sewer Department of Boston. In 1898 he was with the Central Railroad of New Jersey on railroad locations, and in 1899 and 1900, was employed by Albert L. Webster, M. Am. Soc. C. E., Sanitary Engineer, of New York, N. Y., on sewage disposal and sanitary engineering work.

From 1900 to 1903, Mr. Pitkethly was engaged as a Draftsman with the Tax Department, City of New York, and from 1904 to August, 1921, served with the Bureau of Sewers, Borough of Brooklyn, as Draftsman and Assistant Engineer. He collaborated on the design of some of the largest sewers built in the Borough and was in charge of a group of engineers and draftsmen who developed the drainage plans and prepared the contract plans for work costing many millions of dollars. He was an exceptionally rapid worker and was brilliant in his engineering conceptions.

In 1913 Mr. Pitkethly became so ill that his life was despaired of; but after a year he returned to his work. His health failed again in 1921 and he was transferred in August to the Board of Water Supply in the Reservoir Division at Grand Gorge, N. Y. During his stay in Grand Gorge he was connected with the development of work on the Schoharie water-shed as part of the Catskill Mountain Water Supply System, except during part of 1922 when he served as Section Engineer in the Aqueduct Department of the Board of Water Supply on the installation of some additional pipes in the steel-pipe siphons of the Catskill Aqueduct.

Mr. Pitkethly was taken seriously ill after returning, in February, 1926, to Roxbury, N. Y., from the dinner given in New York to the Chief Engineer of the Board of Water Supply. He died at his home in Roxbury, on April 21, 1926, and was buried in Cypress Hills Cemetery, in Brooklyn.

His widow, Josephine A. (Burrucker) Pitkethly, and his two children, Edith Jane and David A. Pitkethly, survive him.

Mr. Pitkethly was elected an Associate Member of the American Society of Civil Engineers on April 6, 1909.

* Memoir prepared by John Charles Riedel, M. Am. Soc. C. E.

CHARLES ENGLISH SHEARER, Assoc. M. Am. Soc. C. E.*

DIED MAY 8, 1925.

Charles English Shearer, the son of William Marion and Margaret (English) Shearer, was born in Indianapolis, Ind., on May 17, 1881. He was educated in the public schools of Indianapolis and, later, entered Purdue University, from which he was graduated in 1905 with the degree of Bachelor of Science in Civil Engineering.

After graduation, Mr. Shearer entered the Bridge Department of the Delaware, Lackawanna and Western Railroad Company, at Hoboken, N. J., where he was engaged in detailing and designing highway bridge crossings and miscellaneous work on elimination of grade crossings in New Jersey.

In 1906, he accepted a position as Checker, with the McClintic-Marshall Construction Company, at the Rankin Plant, near Pittsburgh, Pa. From 1906 to 1910 he was connected with the Memphis Bridge Company, of which he became Chief Engineer in 1907 in charge of engineering designs, including erection of highway bridges, towers, tanks, and structural steel buildings.

In 1910 Mr. Shearer opened an office in Memphis, Tenn., as Structural and Industrial Engineer, in which work he spent the next few years. He supervised the design and construction of the Loosahatchee River Bridge and, in 1911, was engaged in the design and erection of water towers, steel stacks, steel barges, and levee building machines. From 1912 to 1915 he designed and superintended additions to the power plant of the Memphis Street Railway Company, at Memphis, and, as Associate Engineer, also designed a twenty-story office building. Later, in 1916, he had charge of the design and erection of a retort building for the Forest Product Chemical Company, at Memphis.

In 1918 and 1919, Mr. Shearer was Shop Inspector of steel for the United States Government, Ordnance Department Contract T–2, Nitrate Plant No. 1, at Sheffield, Ala., and was especially appointed by the Bureau of Public Works to the Bureau of Insular Affairs, U. S. Government, Ordnance Department, at Manila, Philippine Islands, to design and detail the structural work of the $750 000 Jones Bridge.

On his return, he again took up structural work in the South. During 1920 he reported on, designed, detailed, and supervised the construction of the Second Street Bridge at Clarksdale, Miss., and, in 1922, he was Structural Engineer on the Auditorium Market House, at Memphis. In 1923 he supervised the structural and mechanical design for flood-gates for the Farelly Lake Levee District, Jefferson County, Arkansas. Later, in 1924, he had charge of the re-design and construction of the Submergible and Lift Bridge, at Wisner, La. He also designed and supervised the construction of the Memphis Power and Light Company's Distribution Building at Memphis.

Mr. Shearer had, by his technical ability, personal magnetism, and strict attention to business, reached a stage in his profession, where his services were constantly sought.

* Memoir prepared by J. H. Haylow, M. Am. Soc. C. E.

He lost his life, when the U. S. Steamer *Norman* sank in the Mississippi River, about ten miles south of Memphis, carrying twenty-three of the passengers and crew to their death. He was attending the First Convention of the Engineers of the Mid-South. A meeting was being held at the time in the cabin of the steamer, by local members of the Society, and those resident in the vicinity, for the purpose of forming a Local Section of the Society.

On August 31, 1911, he was married to Annette Ostrander, who, with three minor children, two sons and a daughter, survives him.

Mr. Shearer was elected an Associate Member of the American Society of Civil Engineers on December 6, 1910. He was also a member of the Engineers Club of Memphis.

JOHN McKNIGHT SOUTHGATE, Assoc. M. Am. Soc. C. E.*

DIED JULY 6, 1926.

John McKnight Southgate was born at Rolla, Mo., on September 17, 1881, the third child and only son of Judge W. Wallace and Annie McKnight (Black) Southgate. On his father's side, he was a descendant of a long line of New England ancestors. His mother's parents were among the early English settlers of Missouri, her grandparents having emigrated from Augusta County, Virginia, in 1815, taking three months to make the trip by wagon and flat-boat down the Ohio River.

Mr. Southgate's earlier education was obtained in the Rolla Public Schools with the exception of one year (1896-97) which he spent at St. Mark's School at Portland, Mo. His education was completed at the Missouri School of Mines and Metallurgy at Rolla, where he was a football star and a member of the Sigma Nu Fraternity.

After leaving college Mr. Southgate engaged in railroad engineering. From 1902 to 1908 he was employed by the St. Louis and San Francisco and the Missouri Pacific Railroad Companies, respectively, as Resident Engineer on construction and as Assistant Engineer on location and maintenance. From 1908 to 1911, he was connected with the Sewer Department of St. Louis, Mo., as Engineer of Construction of the River des Peres and other sewers.

In 1911, he organized a construction company of which he was President and engaged in paving and sewer contracting work until 1913, when he returned to private practice as a Consulting Engineer with offices at Rolla. From 1913 to 1923, in conjunction with his consulting practice, Mr. Southgate held the offices of Surveyor and Highway Engineer of Phelps County, Missouri, City Engineer of Rolla, and Resident Engineer of the Missouri State Highway Commission. His consulting practice consisted mainly of municipal and highway work.

In 1923, he returned to the Department of Sewers of the City of St. Louis, in charge of surveys for the $11 000 000 River des Peres Sewer System and remained in that position for one year. In 1924, he was placed in charge of pier construction and harbor improvements in Mobile Bay, for the Mobile and Ohio Railroad Company. He finished this work in January, 1925, and from that time until February, 1926, acted as Division Engineer of Construction and Maintenance of Way, for the same Company.

In February, 1926, Mr. Southgate became connected with the Ozark Pipe Line Corporation (Roxana Petroleum Corporation) as Chief Engineer of Construction. While engaged with his duties in this work he was stricken with acute appendicitis on July 2, 1926, and died four days later in a hospital in Hammond, Ind. His body was taken to his home in Rolla, for interment. He is survived by his widow, Elizabeth Baker Southgate, and one daughter, Mrs. Robert W. Abbett.

* Memoir prepared by Ray Dennison, Esq., St. Louis, Mo., and R. W. Abbett, Esq., Pontiac, Mich.

Mr. Southgate's entire life was spent in the active practice of engineering. He was endowed with those inborn qualities of character so essential to the engineer—sterling integrity, cool and impartial judgment, resourcefulness, and a rare faculty for inspiring confidence and loyalty in his peers and subordinates.

On account of his preference for active field practice he did not always cast his lines in pleasant places, but he bore the privations and discomforts incidental to the accomplishment of his work with unfailing cheerfulness and saw in obstacles only the satisfaction to be found in surmounting them. Capable of infinite patience in attention to the details of a problem, he was quick to reprimand carelessness or laxity in the line of duty, but he was no less ready to recognize and commend worthy effort on the part of others.

The charm of Mr. Southgate's personality made him popular with all classes of people. He was especially loved and respected by those younger men of the profession who were fortunate enough to be numbered among his friends. To several of these he gave the opportunity and assistance which launched them into the practice of engineering.

The place he held in the hearts of his friends can never be filled, but his memory remains as an ideal of the God-fearing gentleman and the engineer.

Mr. Southgate was a member of the American Association for the Advancement of Science and the Missouri Highway Association, having served as President of latter organization in 1919 and 1920. He was also a member of the Executive Committee of the Missouri Good Roads Federation, the organization that promoted and passed the $60 000 000 road bond issue for that State.

He was a member of the Masonic Fraternity and had served as Master of Rolla Lodge No. 213, F. and A. M.; High Priest of Rolla Chapter of Royal Arch Masons; Commander of Rolla Commandery, Knights Templars; Worthy Patron of Rolla Chapter, Order of the Eastern Star; and had also held office in Rolla Council of Royal and Select Masters and Moolah Temple of the Mystic Shrine. Throughout his life he was a devout member of the Protestant Episcopal Church.

Mr. Southgate was elected an Associate Member of the American Society of Civil Engineers on July 11, 1921.

PEYTON BROWN WINFREE, Assoc. M. Am. Soc. C. E.*

DIED MAY 9, 1924.

Peyton Brown Winfree was born at Lynchburg, Va., on September 10, 1868, the son of Major Christopher V. and Virginia A. (Brown) Winfree. All his early life, as well as his later years, was spent in his native city. He was educated in the primary and secondary schools of Lynchburg, completing his academic education at Randolph-Macon College, from which he was graduated as Bachelor of Arts in 1888. He then entered Lehigh University where he earned the degree of Civil Engineer in 1891. Later, he took graduate work leading to the degree of Mining Engineer.

His practical work as an engineer was begun in Pennsylvania, where he was successively engaged in locating pipe lines from Athens to Wilkes-Barre, making preliminary surveys for suburban street railways, and in the location of oil properties. In May, 1894, Mr. Winfree became City Engineer of Bradford, Pa., which position he filled with great credit. The death of his father in June, 1902, compelled his return to Lynchburg, where, notwithstanding the great amount of work necessitated by the administering of large affairs connected with his father's estate, he still had time to devote to his chosen profession.

During 1902 and 1903, he was Consulting Engineer for several coal companies in West Virginia. In 1904, he undertook the location of a pipe line through the mountains for conveying water to Lynchburg, and his skill was such that the city was saved thousands of dollars in its construction. His acumen was so great that he was finally given charge of the construction of the entire water system. In this work he was associated with H. L. Shaner, M. Am. Soc. C. E., Chief Engineer, and James H. Fuertes, M. Am. Soc. C. E., as Consulting Engineer. After the completion of the water supply system at Lynchburg Mr. Winfree undertook a contract for the Atlanta, Birmingham, and Atlantic Railway Company, in Alabama. About 1909, he became Vice-President and General Manager of the Glamorgan Pipe and Foundry Company, where his skill as an engineer was constantly utilized. This position he occupied until the time of his death on May 9, 1924. There was universal sorrow at his death on the part of every young engineer who had been employed under him. He always showed a breadth of understanding and sympathy that completely won their confidence and loyalty.

In 1911, he was appointed Major of Engineers and was assigned to the staff of General C. C. Vaughan, commanding the First Brigade of Virginia Volunteers. His engineering skill was shown in the selection and construction of camps, of which a notable instance is the camp and rifle range at Cape Henry. He resigned his commission on the re-organization of the State Militia into the National Guard.

The Engineering Profession in the State of Virginia will probably remember Major Winfree longest for his indefatigable work in having enacted

* Memoir prepared by C. L. DeMott, Assoc. M. Am. Soc. C. E.

into law the requirements for registration of all engineers. His shrewd common sense, pleasing personality, and intimate knowledge of the need for such a law, coupled with his financial independence, combined to make him incomparable as its promoter. From 1920, when the matter was first brought to the attention of the State Legislature, until March, 1924, when there was finally enacted a workable statute, his influence and means were unsparingly used. Governor Davis honored him by an appointment on the original Board created by this statute for the Registration of Engineers and Architects. He was re-appointed by Governor Trinkle and served until his death.

Peyton Winfree was a fine exemplification of the very best of manhood. His services were always at the disposition of his profession, but were by no means restricted to it, as they were freely given to his neighbors, his city, his country, and his Church.

On the change of the form of the government of the City of Lynchburg in 1920, he was elected as one of the five members of the City Governing Council, and was re-nominated to succeed himself at the spring primary, held just prior to his death.

Major Winfree was one of the District Executives who were engaged in compiling and classifying statistics of the economic resources and industrial activities of the United States, under the direction of the Naval Consulting Board, just prior to the entry of the United States into the World War.

It is given to few men to know of the spiritual life of another man, but surely Peyton Winfree belonged to the Abou Ben Adhem class. No man could have been the open-handed, unostentatious neighbor that he was without having a great deal of the love of God in his make-up. Outwardly, not conspicuously religious, those not connected with his church knew that he was constant in his attendance at religious worship, and he was for years one of the Stewards of the Memorial Methodist Episcopal Church of Lynchburg.

He was a member of the Beta Theta Pi Fraternity, of the Benevolent and Protective Order of Elks, and a Free Mason.

On November 25, 1896, he was married to Mabel Wilbur, of Bradford, Pa. By this union he had three sons and a daughter, Christopher V., Wilbur W., Peyton B., Jr., and Louise Winfree.

Major Winfree was elected a Junior of the American Society of Civil Engineers on October 2, 1894, and an Associate Member on May 3, 1898.

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OF THE

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